THE

RAILROAD ENGINEER'S PRACTICE,

BEING

A SHORT BUT COMPLETE DESCRIPTION OF THE DUTIES OF THE YOUNG ENGINEER IN PRELIMINARY AND LOCATION SURVEYS AND IN CONSTRUCTION.

Fourth Edition. Revised and Enlarged.

BY

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TO

W. H. WILSON, Esq.,

the Chief Engineer under whom the writer began the practice of his profession, on the Pennsylvania Railroad, and whose uniform kindness and interest in his welfare have been continual causes of pleasure and gratitude, this book is respectfully dedicated by T. M. C.

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PRELIMINARY SURVEY.

The Chief Engineer, from an inspection of the various maps of the country he can obtain, and a personal examination of the ground, decides where it will be necessary to run lines to determine which is the cheapest that can be built, having a due regard to the subsequent cost of operation and maintenance, and gives the necessary orders for such lines to the Principal Assistant Engineers.

The method pursued by the Principal Assistant Engineer differs according to the character of the country, and the time that can be devoted to the preliminary survey. A quick, rough method of gaining the requisite information for the location will first be given, and afterward, one more exact—that pursued on the Bennett's Branch Extension of the Allegheny Valley Railroad. The latter is especially recommended where the means of the company will admit of the more accurate work, and where it may not be a matter of policy to begin the construction of the road at the earliest possible moment.

Each Principal Assistant organizes a party which consists as follows: Principal Assistant, Transitman, Leveller, Topographer, Level Rodman, Slope Rodman, Flagman, two Chainmen, three or more Axemen. An Axeman provides a number of stakes in advance and numbers them consecutively from 0, shaving off a smooth place for that purpose, and drives the first one—usually driven flush with the ground,

and called a "plug"—at the place indicated by the Principal Assistant. The latter then starts ahead with the Flagman, and the Transitman sets his transit over the first stake or plug. The Principal Assistant, having decided where he wishes to run the line, sets up the flag. The Chainmen instantly begin chaining toward it, the hind one "lining" the head one, and an Axeman driving the stakes, one every 100 feet, in the order of marking. The Transitman takes his sight, reads only the needle to quarter degrees, records the reading, and starts off for the flag. On arriving there, he sets up ready for another sight, which the Principal Assistant is ready to give him, by waving his handkerchief if the Flagman has not had time to come up. In an open country, the speed of the Chainmen should govern the speed of the party. When there is much underbrush, the Principal Assistant may require several Axemen to clear the way.

The Leveller follows the Transitman as closely as possible, taking levels on every stake, and, if necessary, on abrupt intermediate changes of the slope. His Axeman makes "pegs" (or turning points) and cuts down brush obstructing his view.

The Topographer follows a day behind the Transitman and Leveller. He is provided with a thin box, with a hinged cover on the end, which serves both as a portfolio and a drawing-board. There should be some oiled cloth fastened to it for keeping the paper dry. The paper is tacked to the board with thumb tacks; a convenient size of sheet is 21 × 16 inches. The Topographer has obtained at night from the Transitman and Leveller their notes of the day, and plotted the line on a scale of 400 feet to the inch, noting the elevations at the stations. He takes this into the field with him. His Slope Rodman and Axeman go ahead and measure the transverse slopes, laying a rod upon the ground at each station, and upon it a clinometer;

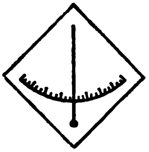
with a tape they measure the distance to where the slope changes, and then measure the new slope and its length, and so on. This is done on each side of the line, and is noted in the Rodman's book in one of two ways: the direction of the slope being indicated either by the signs + and —, or by the inclination of the line dividing the numerator from the denominator of a fraction in which the numerator is the angle, and the denominator is the distance. The notes are given to the Topographer, and with the help of the elevations already obtained from the Leveller, he sketches in the contours. To facilitate this, he uses a table which gives the horizontal distance between two contours taken ten feet apart for each degree, as follows:

1°	is	573	per	10	feet	rise
2°	"	286	"	"	"	"
3°	"	191	"	"	"	"
4°	"	143	"	"	"	"
5°	"	114	"	"	"	"
6°	"	95	"	"	"	"
7°	"	81	"	"	"	"
8°	"	71	"	"	"	"
9°	"	63	"	"	"	"
10°	"	57	"	"	"	"
11°	"	51	"	"	"	"
12°	"	47	"	"	"	"
13°	"	43	"	"	"	"

It is well for him to commit this table to memory.

He estimates distances beyond those measured by the Rodman, and so puts in distant hills, &c. For this it is convenient to have a pocket sextant. The Rodman runs out to different distances, according to the nature of the ground. If the country is level, he may run out 500 feet on each side of the centre-line, only doing so, perhaps, at intervals of 500 feet, or it may be necessary to run out that distance at every station. If the country is hilly, it may be sufficient to run out only 100 feet at each station.

A convenient form of clinometer is formed of a square board, with a string and bullet:



In a new country, where it may be difficult to obtain even a transitman and leveller, the engineer may do all the skilled labor himself by having a vertical circle attached to his transit, reading the vertical angles between successive points, as well as the horizontal ones, and taking the vertical angles at right angles to the line on each side at each transit point. The contours can be calculated by means of a table of latitudes and departures, or otherwise, and plotted in the evening, so that it may be known how the next day's line should be run. The party consists in this case of only two chainmen and an axeman besides the transitman. Even the chainmen could be dispensed with by the use of the stadia rod. Although the proper location of the line can be approximately found in this way, the engineer will have more work to do, the levels especially being liable to error. He ought likewise to have a special aptitude for calculation to perform the necessary calculations in a short time with correctness when fatigued by the bodily and mental exercise of the day.

The more exact method is thus described in a private letter written by Mr. A. B. Nichols, in 1870, when he was Principal Assistant on the Bennett's Branch Extension of the Allegheny Valley Railroad:

"I always run experimental with the *vernier* as follows: Going ahead by myself, I select about the spot where I want to 'plug,' and let the Transitman take a sight on me, setting his vernier to the nearest quarter degree (except in special cases). I

have the head Chainman carry a sight-staff, and set all the stakes with the transit. The head Chainman then sets the fore-sight plug when he arrives at the end of the sight. I use the needle merely as a check on the vernier. I think it better to set the

FORM OF TRANSIT NOTE BOOK.

'H.'

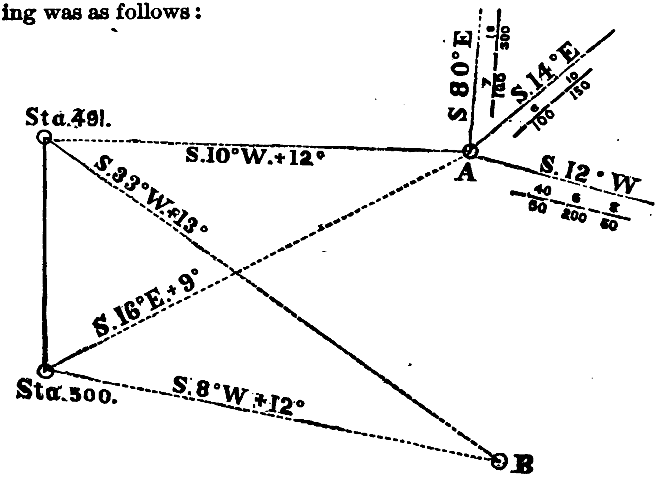
Sta.	Angle.	Deduced Course.	Needle.	Remarks.
8	....	....	....	+ 40 and + 73 edges of turnpike.
7	....	....	....	
6	....	....	....	+ 20 and + 55 stream.
5	L. 6°	N. 32¾° W.	N. 32¾° W.	
4	....	....	....	
3	....	....	....	
+ 30	R. 13°25′	N. 26¾° W.	N. 26¾° W.	
2	....	....	....	
1	....	....	....	
0	....	....	N. 40° W.	

stakes with the transit, as they are more reliable as reference in location, and in an open country they can be set as fast as the Leveller can run (beyond which speed there is no use in running), while in a wooded section there is plenty of time to set them while the Axemen are clearing. In thick woods, the Principal Assistant's voice has often to be taken as the guide ahead. The bench marks should be marked with the number of the station immediately preceding, and the distinctive letter of the line. Thus, if there happen to be a bench at 7 + 60 of 'H' line, it should be marked B. M. | 7 'H.'

"In regard to the Topographer's duties, I do not like the system of putting in the topography in the field. It has always been the custom, I believe, to run experimental one day and locate over the next. Mr. J. A. Wilson's method differs somewhat from this, and, I think, with reason. Topography put in in the haste that is inevitable in the field, is liable to many errors, and locations made on the previous day's experimental may not suit the country ahead. Mr. Wilson's method is to run all the necessary lines, take all the necessary notes, and then go into office quarters and work the maps up, and make a paper location which may then be run in and modified in the field.

"In 'Morrison's Cove' Mr. Linton took charge of the topo-

graphical department, taking the topography notes himself. His instruments were: a pocket compass, mounted on a light tripod, a Locke's level, and a small slope-board. His method of proceeding was as follows:



"Say that A is a tree on a hill, and B another point on another hill. He would set his compass up at 491, for instance, take the courses to A and B, and measure the vertical angle with his slope-board. He would then proceed to A and take slopes in all directions, and in like manner from B, using his slope-board and level for heights and slopes. Then going to another station, as 500, he would fix the points A and B by other courses and slopes. Hollows can be shown by running a course up and taking slopes to right and left. By that means he could show the topography sometimes a half a mile from the line. I have known his elevations, say at A, to come within a foot of each other at the distance of half mile from the line, deduced from vertical angles taken with the slope-board, as from 491 and 500; seldom over two feet difference.

"The slope-board is a modification of the square-board and bullet. It will read to quarters of a degree, is furnished with sights,* and is used as follows:

"The Assistant Topographer takes his stand at the station, and

* Mr. Linton's improved slope instrument may now be obtained at mathematical instrument makers.

gives the right angle to the line by means of a right-angle box, or otherwise. The Slope Rodman measures out the horizontal distance with a ten-feet-long pole to change of slope, and sights back on the man at the station, taking a point on the other's person (previously determined) at the same height above ground as his own eyes. He reads the slope, calls it and the distance out, and in the meanwhile the man at the station, be it the Assistant or the other Rodman, checks the slope by sighting on his person. Rodman No. 1 then measures ahead to the next change, while Rodman No. 2 comes up to *change* No. 1; they measure the slope, and so on. The Assistant keeps the books, and should be furnished with a 'Jacob's staff' and compass for taking buildings, and while so engaged the Rodmen can measure the sizes of said building with a tape, or can go on taking slopes, which they afterward report to the Assistant. Slopes should never be estimated, except one at the end of a series, and then it should be so marked, and the contours derived from it should be dotted on the map to avoid errors in location. In taking short slopes, one Rodman can take the right and the other the left of the line, thus facilitating matters."

From an inspection of the maps, it will be seen on which routes it is necessary to have paper locations made. A paper location is such a line drawn upon the plan as may appear, taken in connection with the profile, to require the least excavation and embankment. The following is an excellent method of obtaining the cheapest location on the preliminary map: Having located a trial line by inspection, a profile is made, and grades assumed and drawn. A horizontal plane is supposed to pass through the point on the grade line at each station, and a point, in its line of intersection with the ground surface opposite the station, is marked in red. Having plotted these red points for a sufficient distance, they are connected by a line, which will resemble a contour line, and actually becomes one when the grade is level. The nearer the paper location can be drawn to this line, the less will be the excavation and embankment. If it coincides with it, the line will be a surface line.

Having made the locations on such lines as are considered desirable, a new profile is made from an inspection of where the located line cuts the contours, and cross-sections are plotted on a scale of ten feet to the inch, or on Trautwine's cross-section paper. From these cross-sections, the amounts of excavation and embankment are calculated, and the results embodied in a table of the following form :

Sta.	Distance.	Eleva.	Grade.	Cut.	Fill.	Areas.			Solids.		
						Excavation.		Embankment.	Rock.	Earth.	Emb.
						Rock.	Earth.				

The slopes of the cuts are sometimes assumed as follows :

When the slope of the ground is 20° or less, make the slope 1½ to 1.

When the slope of the ground is 20° to 35°, make the slope 1 to 1.

When the slope of the ground is over 35°, make a vertical wall.

Rock stands at ¼ to 1.

Embankment is generally taken as sloping 1½ to 1.

From the calculated amount of excavation and excess of embankment, an estimate is made of the costs of different routes, and it is thus found which lines it will be necessary to actually locate in the field, in order to obtain a closer estimate, or for the purpose of constructing.

In crossing a mountainous country, the Chief Engineer often decides which of several passes it may be necessary to have lines run through, by observing their heights with an Aneroid barometer. Those, of course, are thrown out of the number requiring a more exact determination, which have a higher summit, without any compensating advantages in the way of the cost of the work. The

Aneroid is first read at some point of known elevation, and then taken to the summit whose elevation is desired; the difference of elevation in feet is given by the following formula:

$$60664\left(1+\frac{T+t-64}{901}\right)\log\frac{H}{h}$$

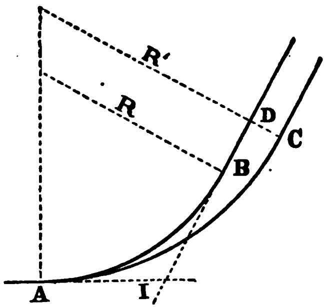
in which H and h are the heights in inches of the barometer, at the lower and upper stations, and T and t are the temperatures of the air in degrees Fahrenheit at the times of observation at the same stations.

LOCATION.

The locating party is organized somewhat differently from the preliminary one. We have: Principal Assistant, Transitman, Leveller, Level-Rodman, Front Flagman, Back Flagman, two Chainmen, two or more Axemen.

The axeman who drives the stakes now carries tacks, or better, lath nails, with him, and drives one in the plug at the point where the Transitman is to set up. The Transitman uses the vernier entirely, not using the needle, unless as a check on the tangents. The curves are all run on the ground, and the stakes which come upon them set with the transit. The Leveller keeps close up to the Transitman, and constantly reports the heights to the Principal Assistant. The tangents are generally fixed by the paper locations, and the usual object is to run a given curve from one end of a tangent, and strike the hill with the point of tangent ($P.\ T.$) at a given elevation, viz., that on the paper location. Other problems will also often arise. The following are the most useful:

Problem 1. To change a curve so that it shall come out in a parallel tangent at a given distance from the old tangent, by changing the radius. (From "Haslett & Hackley's Pocket-Book.")



To change the curve $A\ B$ so that it shall come out at C,

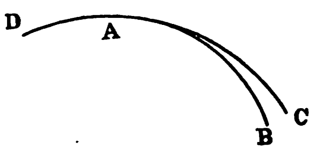
$$R' = R \pm \frac{D\ C}{1 - \cos.\ I};$$

or otherwise ;

$$\text{Degree of curve } A\ C = \text{degree of curve } A\ B \mp \frac{8\ D\ C}{7\ n^2}$$

in which n is the number of 100-feet chords in $A\ B$.

Problem 2. To change the origin of a curve so that it may pass through a given point.



To move A, the point of compound curvature, so that the curve $A\ B$ will pass through the point C.

Take the distance $B\ C$, divide it by $A\ B$, and multiply by 57.3, and we get the difference in deflection $C\ A\ B$, which, divided by the number of stations in $A\ B$, gives the difference in deflection per station (or look in the table of natural sines for $\frac{B\ C}{A\ B}$, from which is obtained the angle $C\ A\ B$, which is to be divided by the number of stations). Then take the difference between the degrees of the curves $A\ B$ and $D\ A$; then, the difference between

the degrees of the curves is to the difference in deflection per station as $A\ B$ is to the number of feet forward or backward we must go, on the curve $A\ D$, to strike the point C.

This is expressed by formula as follows:

$$x = \frac{R\ r\ d}{l\,(R - r)}$$

in which R and r are the radii of the curves, l is the length of the last curve and d is the distance the point of tangent is to be moved over; x being the distance the point of compound curvature is to be moved.

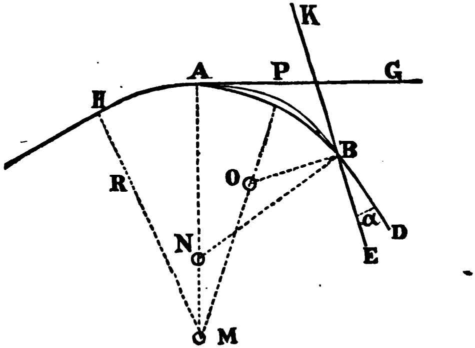
Problem 3. Having located a compound curve terminating in a tangent, it is required to change the point of compound curvature so that the curve will terminate in a tangent parallel to the located tangent, at any required distance perpendicular thereto. Divide the required distance between parallel tangents by the difference of radii of the two last branches of the curve. From the cosine of total amount of curvature in the last branch subtract or add this quotient. The remainder, or sum, will be the natural cosine of the amount of curvature required for the last radius.

This may be expressed by a formula as follows:

$$\cos.\ (I - x) = \cos.\ I \pm \frac{d}{R - r}$$

in which I is the total angle in the second branch of the curve, R and r are the radii, d is the distance the tangent is to be moved over, and x is the angle by which the total angle in the last branch is to be increased or diminished (the total angle of the first branch being equally diminished or increased). If the radius of the first curve is larger than that of the second, the upper sign is to be used for moving the curve forward, and the lower one backward; if the radius of the second curve is longer than that of the first, the upper sign will move it backward, and the lower one forward.

Problem 4. Having located a compound curve terminating in a given tangent, it is required to change the point of compound curvature, and also the length of the last radius, so as to pass through the same terminating point with a given difference in the direction of the tangent.



Having the curve $H\,A$ and the curve $A\ B$, with tangent $B\ D$. it is required to continue the first curve from A to such a point, P, that the tangent at B will have the direction $B\,E$.

Continue the curve $H\,A$ to the point P, given by the following equation:

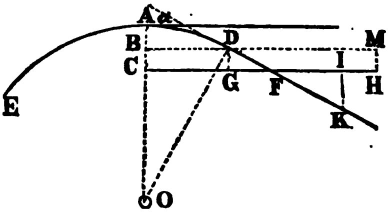
$$Cotan.\ \tfrac{1}{2}\ A\ M\ P = \frac{R}{R'}\ (cotan.\ \tfrac{1}{2}\ A\ N B + cotan.\ \tfrac{1}{2}\ \alpha) - cotan\ \tfrac{1}{2}\ \alpha.$$

The curve from P to B is, of course, found by measuring the total deflection angle, and dividing by the number of stations.

We can, by means of this problem, connect two curves running toward each other with a third one. Let A and B be points on the respective curves. We wish to continue the curve $H\,A$ past A to some point, P, from which to run some third curve connecting with the other curve at B (the tangent $B\,E$ being common to the last two curves).

Measure the angle $G\ A\ B = \tfrac{1}{2}\ A\ N B$; also the angle $K\ B\ A = \tfrac{1}{2}\ A\ N B + \alpha$; and the distance $A\ B = 2\ R'\ sin.\ \tfrac{1}{2}\ A\ N B$. Then calculate $A\ M\ P$ from the above equation; dividing by the degree of the curve $H\ \ A$ gives the distance $A\ \ P$ in stations. From $P\ \ O\ \ B = A\ N B + \alpha - A\ M\ \ P$ and the distance $P\ \ B$, we can find the degree of the curve $P B$.

Problem 5. To change the radius of a curve so that it will come out in a given tangent.



To change the radius of the curve $E\ D$ so that it will come out in the tangent $C\ H$.

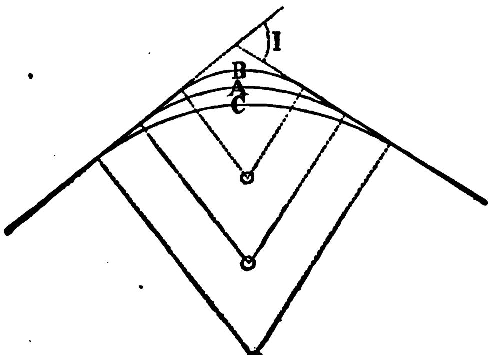
Having run the curve until the tangent $D\ K$ is nearly parallel to $C\ H$, measure the offsets $D\ G$ and $I\ K$, and the distance $G\ I$. Calculate $G\ F$ and then $\alpha \left(= tan.^{-1} \dfrac{D\ G}{G\ F} \right)$. We could also measure this angle directly by measuring off $M\ H = D\ G$ and taking a sight on M. Then $A\ C = A\ B + B\ C = R$ *ver. sin.* $\alpha + D\ G$.

We then find the new radius by Problem 1.

$$R' = R \pm \frac{A\ C}{ver.\ sin.\ I}$$

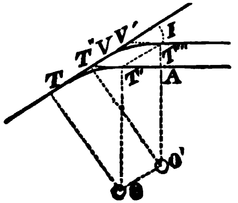
($I =$ the former total angle minus α).

Problem 6. Having located a curve connecting two tangents, it is required to move the middle of the curve any given distance either toward or from the vertex.



$$R' = R \pm \frac{A\ B \text{ or } A\ C}{\dfrac{1}{\cos.\ \frac{1}{2}\ I} - 1.}$$

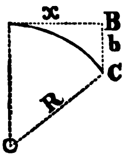
Problem 7. To change the origin of a curve so that it shall terminate in a tangent parallel to a given tangent at a given distance from it.



Let $T\,T'$ be the curve, $V\,A$ the given tangent, and $V'\ T'''$ the parallel tangent.

$$\text{Then } T\,T' = \frac{A\,T''}{sin.\ I}$$

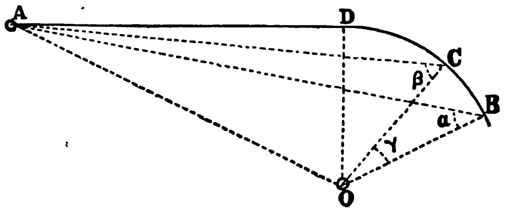
Problem 8. To find how far back it is necessary to go from the point B, to strike the point C with a curve of given radius; $B\,C$ being known.



$$x = \sqrt{b\,(2\,R - b)} = \sqrt{b\,D}$$

approximately, where D is the diameter of the curve.

Problem 9. To draw a tangent to a curve from a point outside of it.

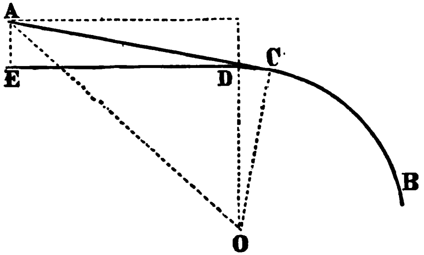


$$Tan.\,B\,A\,O = \frac{sin.\ \alpha}{\left\{sin.\,(\alpha+\gamma) - sin.\ \alpha\right\} cot.\,(\alpha+\gamma-\beta) - cos.\,(\alpha+\gamma)}$$

$$sin.\ D\ A\ O = \frac{sin.\ B\ A\ O}{sin.\ \alpha}$$

$$D\,O\,C = 90^\circ + D\,A\,O - B\,A\,O - \gamma - \alpha.$$

What A is near C, and the curve has been run too far around, the following simple solution of this problem is given by Mr. J. S. Dunning, in the *Railroad Gazette* for March 17, 1882 :



Measure $D\,E$ and $E\,A$; then

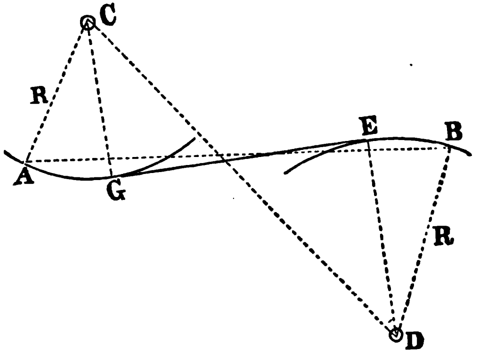
$$tan.\ A\ O\ D = \frac{E\ D}{R + A\ E}$$

$$cos.\ C\ O\ A = \frac{R\ sin.\ A\ O\ D}{E\ D}$$

$$D\ O\ C = C\ O\ A - A\ O\ D.$$

In practice, however, $A\ E$ is generally very small in comparison with $E\ D$, and it is then a sufficient approximation to turn the transit when at D on A, and measure the angle $A\ D\ E$, and make $D\ O\ C = A\ D\ E$.

Problem 10. To draw a tangent to two curves already located. 1st. When the curves are in opposite directions.



Stop both curves before getting to the tangent points. Ob-

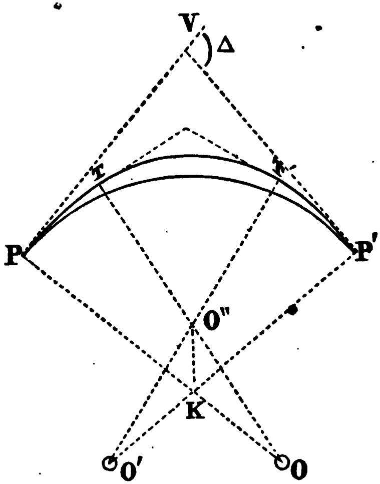
serve BAC and ABD, and measure AB. In the triangle ABC, calculate CB, CBA and ACB (AC, AB and CAB being known). In the triangle BCD calculate CD, BDC and DCB (BC, BD and $CBD = CBA + ABD$, being known).

$$Cos.\ EDC = \frac{R + R'}{CD.}$$

$$BDE = BDC - EDC$$
$$ACG = ACB - (EDC + DCB).$$

2d. When the curves are in the same direction. This is an extension of Problem 4.

Problem 11. To substitute a compound curve for a simple one. ("Henck's Pocket Book," p. 59.)



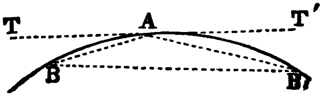
Let $PK = P'K = R$ and $PO = P'O' = R'$ and $TO'' = T'O'' = R''$ and $POT = P'O'T' = 2\theta$ and $TO''T' = 2\theta'$. Assume R' and θ'.

Then $\Delta = 4\theta + 2\theta'$ and $O'O'' : KO' :: sin.\ O'KO'' : sin\ KO''O'$,

or $R' - R'' : R' - R :: sin.\ (\pi - \frac{1}{2}\Delta) : sin.\ \theta'$.

$$\therefore R' - R'' = \frac{(R' - R)\ sin.\ \frac{1}{2}\Delta}{sin.\ \theta'.}$$

Problem 12. To locate the second branch of a compound curve from a station on the first branch.

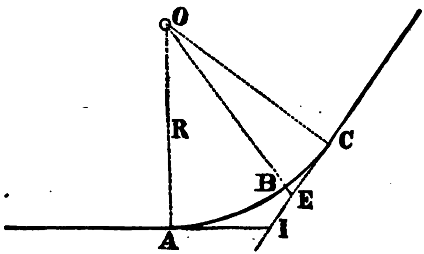


Let $A\,B$ be the first branch of a compound curve and D its deflection angle, and let it be required to locate the second branch $A\,B'$, whose deflection angle is D', from some station B on $A\,B$. Let $n =$ the number of stations from A to B, and $n' =$ number of stations from A to B'. Let $V = A\,B\,B'$; $(B\,A\,T = n\,D)$

$$V = \frac{n'\,(n\,D + n'\,D')}{n + n'}$$

(See "Henck's Pocket Book," p 61.)

Problem 13. To locate a tangent from an inaccessible point on a curve.

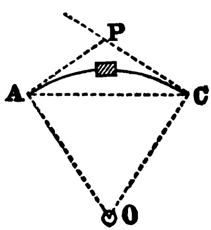


Let C be the inaccessible point. Run the line to a point B.

$$B\,E = R\left(\frac{1}{\cos.\ C\,O\,B} - 1\right).$$

$$B\,E\,C = 90^\circ - C\,O\,B.$$

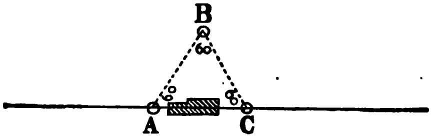
Problem 14. To pass an obstacle on a curve.



1st method. $A\,C = 2\,R \sin \tfrac{1}{2}\,A\,O\,C.$
2d " $A\,P = R \tan. \tfrac{1}{2}\,A\,O\,C.$

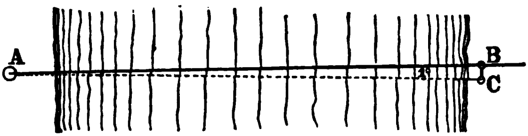
$A\,O\,C$ must be assumed at such a value as, it is supposed, will carry the line beyond the obstacle.

Problem 15. To pass an obstacle on a tangent. ("Mifflin on Railway Curves," Prob 17.)



$A\,C = A\,B = B\,C$

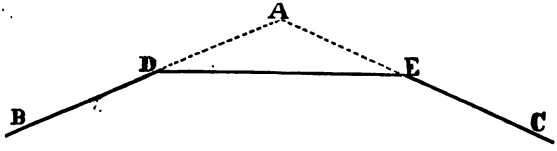
Problem 16. To find the distance across a river in a preliminary survey.



From A put in the plug B on the opposite side in the line which is being run. Then turn off one degree and put in the plug C. Measure the distance $B\,C$; then

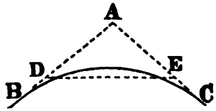
$$A\,B = \frac{100\,B\,C}{1.75} \text{ or } = 57.3\,B\,C.$$

Problem 17. To find the radius of a circular arc which shall successively touch three straight lines $B\,D$, $D\,E$ and $E\,C$. (From "Rankine's Civil Engineering.")



$$\text{Radius} = \frac{D\,E}{\tan. \tfrac{1}{2}\,D + \tan. \tfrac{1}{2}\,E} \quad (D = A\,D\,E \text{ and } E = A\,E\,D.)$$

Problem 18. To connect two tangents with a curve of a given radius when the point of intersection is inaccessible. (From "Rankine's Civil Engineering.")



$$D\,A\,E = 180° - (A\,D\,E + A\,E\,D)$$

$$A\,D = D\,E\,\frac{\sin.\ A\,E\,D}{\sin.\ D\,A\,E};\quad A\,E = D\,E\,\frac{\sin.\ A\,D\,E}{\sin.\ D\,A\,E.}$$

$$D\,B = R \text{ cotan. } \frac{D\,A\,E}{2} - A\,D;\quad E\,C = R \text{ cotan. } \frac{D\,A\,E}{2} - A\,E.$$

The transit points (marked *Tr. P.*) are called "plugs," and consist of stakes driven flush with the ground. They are guarded by a stake set on one side with the number turned toward the plug, and under it written (say) "3′ off." All the other stakes should have their numbers turned toward the beginning of the line.

The following is the form of field-book: (From Shunk.)

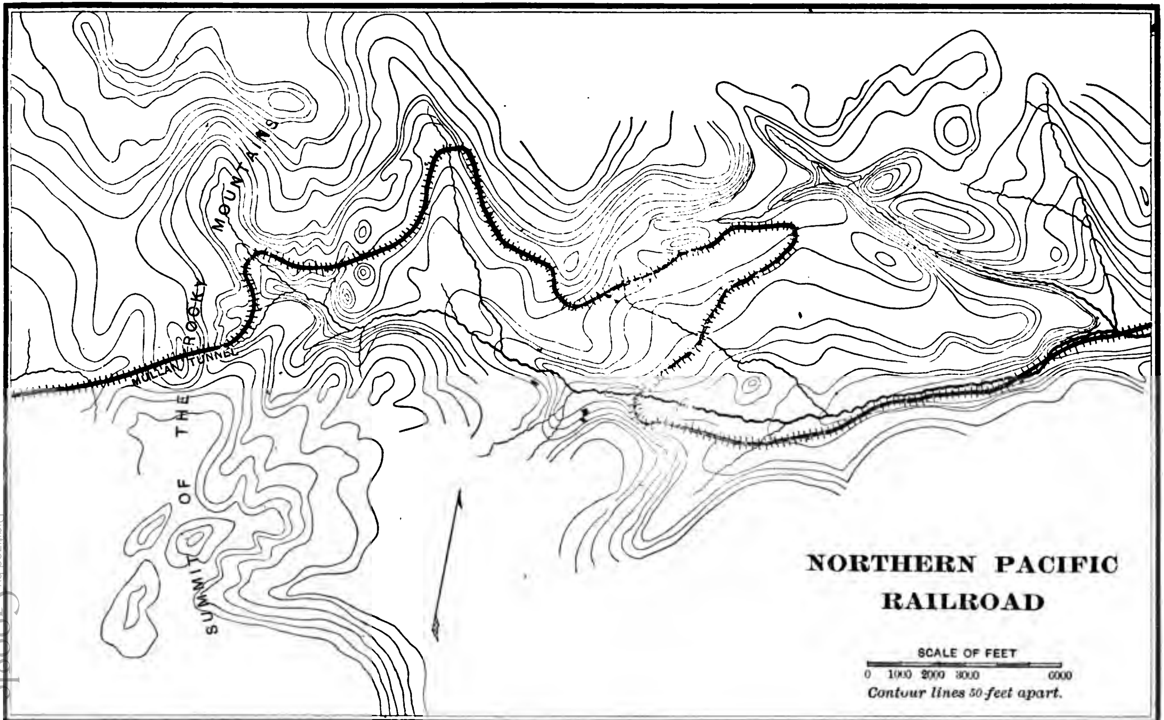
Station....	Distance....	Deflection..	Index......	Tangent...	Course.....	Mag. Course.	Remarks...
23 O	100	......	......	......	N.20°00′W.	N.20°05′W.	At sta. 24 + 50 commence a 4° curve to the L. for 35°12′.
24	50	......	......	......	....	....	
P C. + 50 O	50	1°00′	1°00′	.....	....	....	
25	100	2°00′	3°00′	......	....	....	
26	100	2°00′	5°00′	......	....	....	
27	100	2°00′	7°00′	......	....	....	
28 O	100	2°00′	16°00′	14°00′	....	N.34°07′W.	
29	100	2°00′	18°00′	......	....	....	

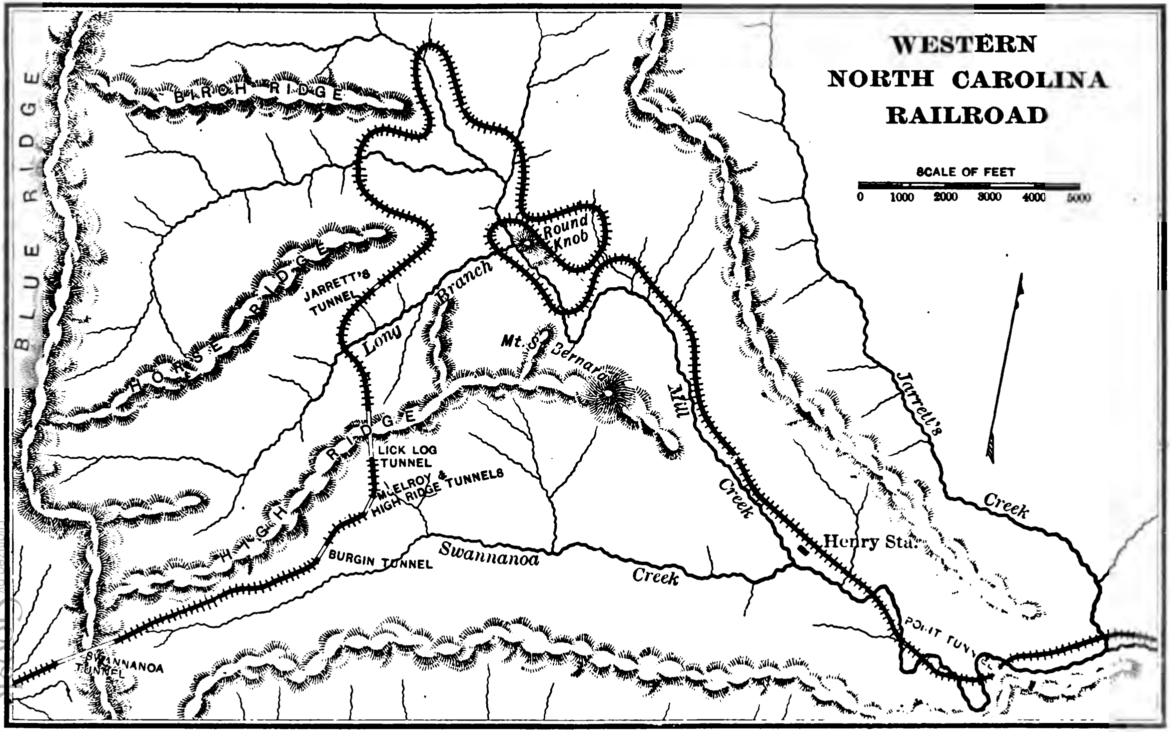
In locating in a mountainous country, it will generally be more conducive to economy to throw the heaviest grades in one part of the road, if possible. Auxilliary engines can

then be used on those portions to assist in hauling the trains, so that the motive power can be more accurately adjusted to the size of the trains, and it will not be necessary for an engine to haul only a portion of its maximum load for a long distance. For example, the grades on the Pennsylvania Railroad in crossing the Allegheny Mountains, going west, are principally condensed into the portion between Altoona and Gallitzin, a distance of twelve miles, with a maximum of 100 feet per mile, while from Harrisburg to Altoona, a distance of 132 miles, the maximum is only 21 feet per mile.

In a line across the principal water-ways of a country, it will generally be better to make it undulating between the ridges and valleys than to go to great expense in making deep cuts and heavy fills to secure a level grade; for the interest on the increased capital account will amount to more than the extra cost of the motive power required to move the trains. The action of gravity, too, on the downward portions may save a part of the fuel which would be required on a level.

If a valley runs in the general direction of the road, it will generally be better to follow it rather than to take the ridge between adjacent valleys. It is found that valleys are usually composed of long pieces of gentle slope connected by a sudden rise, that is to say, of pools and falls or rapids in the stream. At such sudden rises developments can be made, if necessary, to obtain the ascent with a limited maximum grade. On the St. Gothard Railroad it was at these points that the helical tunnels were cut in the rock, the line emerging and crossing itself by a bridge at the height gained by the length of the helix. When developments have to be made, the best places are where, in the curve of the valley, the concave side is steep while the other is a flat point which can be well utilized for the curves. It is a peculiarity of curved valleys to have lower


ROCKY MOUNTAINS
MULLAN TUNNEL
SUMMIT OF THE
NORTHERN PACIFIC
RAILROAD
SCALE OF FEET
0 1000 2000 3000 6000
Contour lines 50 feet apart.


WESTERN
NORTH CAROLINA
RAILROAD
SCALE OF FEET
0 1000 2000 3000 4000 5000
BLUE RIDGE
BIRCH RIDGE
HORSE RIDGE
HIGH RIDGE
JARRETT'S TUNNEL
Long Branch
Round Knob
Mt. St. Bernard
LICK LOG TUNNEL
McELROY & HIGH RIDGE TUNNELS
BURGIN TUNNEL
SWANNANOA TUNNEL
Swannanoa Creek
Mill Creek
Jarrett's Creek
Henry Sta.
POINT TUNNEL

ground on the convex side; what has been eroded from the concave side has been deposited on the other.

There is appended a map of the location of the Northern Pacific Railroad over the Rocky Mountains, made in 1887 by Mr. J. C. Chesbrough, which deserves study. The reader is referred to the larger plan and description in the *Journal of Engineering Societies* for June, 1884. It is recommended to the student to make a profile of the located line from the map and to rule grades. A plan is also given of the location of the Western North Carolina Railroad over the Blue Ridge, made by Mr. H. Eaton Coleman in 1881. This would be more interesting if it had the contours drawn, but it is not known whether such a map can be procured. It is reduced from a lithograph where only hatchings were used to represent the hills. It will be noticed how much advantage has been taken of lateral valleys to increase the length of the line. The maximum grade is 116 feet per mile reduced on curves at the rate of one-tenth per hundred for each degree. The maximum curve is 10°. For another interesting example of location, see the *Railroad Gazette* of Nov. 27, 1885.

The maximum grade and minimum radius of curvature to be adopted depend on the judgment of the Chief Engineer, based on the probable amount of traffic the road will command. On the Hudson River Railroad the sharpest curve is 3 degrees; on the Pennsylvania it is 6 degrees, except in the mountain division, between Altoona and Gallitzin, where a $10\frac{1}{2}$ degree curve is employed in one instance. On the Callao, Lima & Oroya Railroad, across the Cordilleras in Peru, the sharpest is one of 120 metres radius, or a 14 degree curve. The maximum grade on the Pennsylvania Railroad is likewise on the mountain division, and amounts to $1\frac{8}{10}$ per cent. On the Oroya Railroad it is 4 per cent. On the St. Gothard Railroad, in Switzerland, the minimum radius is 280 metres, and the maximum grade $2\frac{7}{10}$ per cent.

The heaviest grade, however, is not usually combined with the sharpest curve. On the construction of the mountain division of the Pennsylvania Railroad in 1853, the maximum grade of $1\frac{8}{10}$ per cent. was only used on tangents, and on curves it was reduced .025 per cent. for each degree of curve. For example, the grade on the $10\frac{1}{2}$ degree curve is only 1.6375 per cent. On the Oroya Railroad, on curves of radii between 120 and 300 metres, 3 per cent. grade was allowed; but from curves of 300 metres radius to tangents 4 per cent. was the maximum.

The Pennsylvania Railroad rule may be put in the form of an equation as follows:

$$\text{Grade in feet per 100 on a curve} = \text{max. grade on tangent} - \frac{\text{degree of curve}}{40}$$

If this is preferred in terms of the radius, for

$$\frac{\text{degree of curve}}{40} \text{ write } \frac{143}{R}$$

This does not seem to give a sufficiently great reduction to accord with modern experiments. Some engineers think that the reduction should be as much as $\frac{1}{10}$ instead of $\frac{1}{40}$ per each degree of curvature. Perhaps $\frac{1}{20}$ is the best fraction to use. In Bavaria, experiments gave a formula deduced by Herr von Röckel, which, reduced to English measurements, is:

$$\text{Grade in ft. per 100 on a curve} = \text{max. grade on tang.} - \frac{213}{R - 180},$$

R being in feet.

Mr. H. G. McClellan found by experiment, on a railroad of 3 feet gauge with short trains, that this formula equalized the resistances satisfactorily.

The principal resistance on curves is due to the enforced parallelism of the axles in a truck, and is inversely proportional to a function of the radius, as expressed by these formulas. The use of the bogey truck, with a smaller

wheel base than on European carriages, makes the American cars offer less resistance. For the former, therefore, a greater reduction would be necessary. It has been attempted to keep the axles always radial to the curve, but the devices do not seem to have been a practical success. The other resistances on a curve are two: 1st, that due to the difference in length of the outer and inner rails, one wheel having to slip or slide by this amount when rigidly attached to the axle; and 2d, that due to the line of traction being on a chord of the curve, and not parallel to it. This latter resistance is greatest when the engine is starting from rest, and is counteracted by the centrifugal force. As, however, the tractive force exerted between adjacent cars diminishes from the front to the end of the train, it is only possible to neutralize the centripetal component of the *average* tractive force with the centrifugal force. A simple calculation will show that this may be accomplished, when the cars are 60 feet long, with a speed of thirty miles per hour. Leaving out of the consideration the elevation of the outer rail, which is especially necessary to prevent the engine from leaving the track, the effect on the front cars, when the centrifugal force is balanced by the centripetal component of the average tractive power is, to draw them towards the inside rail, while the last cars will be thrown against the outer rail. This effect will be the greater the longer the train, as will be likewise the resistance of the train due to the enforced parallelism of the axles. The late Mr. Morley considered that the reduction of the grade on curves should depend on the "ruling" or maximum grade on the division. His observations of the proper amount may be expressed by the formula:

$$\text{Grade in ft. per 100} = \text{max. grade on tang.} - \frac{\text{degree of curve}}{12 + 6 \text{ max. grade}}$$

Further experiment is needed, though it is obvious that as more powerful engines are introduced, the length of

train will be increased, and any formula will have to be accordingly modified. The formula deduced by Mr. W. W. Coe and Mr. C. C. Wentworth on the Norfolk & Western Railroad from experiments, where the maximum grade was 2 per cent. and the maximum train length on these grades was 600 feet, is :

$$\text{Grade in feet per } 100 = \text{max. grade on tangent} - \frac{\text{degree of curve}}{100} \times \text{length of curve in stations.}$$

It will be seen that it is an advantage to shorten the length of the train, and the prevailing tendency to increase the carrying capacity of the cars is in the right direction, and should only be stopped on roads having a large constant traffic, by their equalling when loaded the weight of the engines. Probably the reduction of the grade on curves would then be less than that recommended above.

At the foot of heavy mountain grades, a reverse grade is sometimes put in for a short distance, to stop cars that may have become accidentally detached from the train, so that they shall not run into trains farther down the line.

When it is necessary to curve from one direction to another, a short piece of tangent should always be interposed to enable the proper elevation of the outer rail to be secured. On the St Gothard Railroad 40 metres is required in the specification. In the United States it is sometimes thought that two rails, or about 60 feet, is sufficient. Of course, the proper length depends on the sharpness of the curves, and the greatest permissible grade. (See remarks on a subsequent page in regard to curves of adjustment in connecting a curve and tangent; when they are used there is no necessity for a piece of tangent between reversed curves, although it is always better to have it.)

For a temporary track, with a gauge of 4 feet 8½ inches and ordinary engines, a curve of 187 feet radius may be

made, round which the engine will easily go; on account of the danger of running off when going at great speed, and likewise on account of the great wear of both rails and tyres, such a curve should only be a temporary one. In *Engineering News* of Oct 2, 1880, a curve of 90 feet radius is mentioned, around which freight engines constantly go, this being likewise the minimum radius of the New York Elevated Railroads, and in the "Transactions of the American Society of Civil Engineers," vol. VII., p. 107, a curve of 50 feet radius is mentioned, around which a very large traffic was conducted during the late war. The outer rail was elevated about eight inches above the inner rail, and the train was run at eight to ten miles per hour.

After starting, cars will just run on a grade of .5 per 100. When new they will become slightly accelerated in velocity. They will start themselves on a grade of about .7 per 100 when in good order. On the Pennsylvania Coal Company's tracks, which were worked by gravity, the grade for loaded cars was fixed at 45 feet per mile and for empty cars at 47 feet. This gave a speed of about 25 miles per hour on the tangents, though each train carried a brakeman to limit the speed, if necessary, by the brake.

The final location having been made, the line is divided up into lengths of about one mile each, called sections, and a board placed on end at the dividing station, with the numbers of the sections on the sides. An estimate is made of the amount of earth-work and masonry on each section, and the road is advertised for contract. The contractors are each furnished with a printed copy of the quantities in each section, and allowed to take such notes as they require from the map and profile, and walk over the ground, the section boards guiding them to the different work.

CAMP EQUIPAGE.

Through the courtesy of Mr. E. T. D. Myers, the following list of articles required for "camping out" in preliminary and location surveys is inserted. It has been derived from an extensive experience:

One light wagon and horse, two two-horse wagons and horses, harness, etc., one saddle and bridle, five halters, five horse blankets, three wall tents and flies, 9 feet by 9 feet, with 3 feet walls and 8 feet high; one house tent, 8 feet by 8 feet, 8 feet high; four Indian rubber tent floors, twelve camp stools, two army combs and brushes, two lanterns, six candlesticks, one box of candles, one mess chest, one table, 4½ by 3 feet; one table, 6 by 3 feet; twenty large tent pins, one small grindstone, one spade, two water buckets and dippers, two horse blankets, one cook's bucket, one box of soap, twelve boxes of blacking, two blacking brushes, four mattresses, one box of paints, six leather straps, two frying pans, one spider, one gridiron, one gill measure, one rolling pin, one weighing scale, one gallon measure, one quart measure, one oven, one skillet, one pot, one tea kettle, one coffee pot, one pair of pot hooks, two tin pans, one bread pan, twelve tin cups, twelve tin plates, one tin washing pan, four tin basins, one pepper box, twelve coffee mugs, twelve plates, one dozen knives, forks, teaspoons and tablespoons, one large iron fork and one large iron spoon, one wooden tray, one molasses pitcher, one sugar dish, one butter dish, one two-gallon jug, one one-gallon jug, one coffee mill, one sifter, one dozen towels, one large dish, one coffee box, one sugar box, one lard box, one large pitcher, one large tub, one flat-iron (one medicine chest, with bitters, ginger, quinine, oil, oil of cloves, brandy, watch-maker's oil, laudanum, salve, bandages and lint, No. 6, calomel, horse fleam, lancet, hartshorn), smoking tobacco, pipes, chewing tobacco, one coil of small rope,

two bunches of twine, one large twine needle, three small stoves, a half dozen papers of tacks, a half gross of matches, musquito bars, one dozen axes, three dozen transit books, four dozen level books, profile and mapping paper, tracing muslin, pens, knife, mouth glue, India ink, common ink, letter book, red ink, rubber for water colors, pencils, foolscap, letter paper, cartridge paper, chalk, red flannel, map cases of tin, box for stationery, drawing board, color cups and brushes, transit, level, rod and target, chain, tapes, transit rods, chain pins, triangles and rulers, drawing scales, hand axes and belts, brush hooks, small hatchet, two saws and saw files, six pounds of nails.

If provisions have to be carried, the following may be of use· According to the United States Army Regulations a ration is twelve ounces of pork or bacon or canned beef (fresh or corned), or one pound and four ounces of fresh beef, or 22 ounces of salt beef; eighteen ounces of soft bread or flour, or sixteen ounces of hard bread, or one pound four ounces of corn meal, and to every 100 rations fifteen pounds of beans or peas, or ten pounds of rice or hominy; ten pounds of green coffee, or eight of roasted (or roasted and ground) coffee, or two pounds of tea; fifteen pounds of sugar, four quarts of vinegar, four pounds of soap, four pounds of salt, four ounces of pepper, one pound eight ounces of adamantine or star candles, and to troops in the field, when necessary, four pounds of yeast powder to 100 rations of flour.

The travel ration is: For 100 rations, 112½ pounds of soft bread or 100 pounds of hard bread, 75 pounds of canned fresh beef or 75 pounds of canned corned beef, 33 one-pound cans baked beans, or 20 two-pound, or 15 three-pound do.; 8 pounds roasted coffee, 15 pounds sugar. Six-pound cans of beef or three-pound cans of beans should be habitually issued. One and two-pound cans of beans and two and four-pound cans of beef will be issued when

it is not convenient to provide the larger ones, or when small amounts are demanded.

The forage ration is fourteen pounds of hay and twelve pounds of oats, corn or barley ; for mules, the same amount of hay, but only nine pounds of oats, corn or barley. For calculating the measure of the latter, one bushel of oats weighs 24, one bushel of corn weighs 56, and one bushel of barley weigh 48 pounds. Pressed hay weighs 11 pounds per cubic foot and is put up in bundles weighing about 300 pounds.

The following is the list of supplies sent in the Collins engineering expedition to Brazil: 225 mattresses, 225 pillows, 225 pillow-cases, 200 brown blankets, 50 barrels extra mess beef, 35 barrels mess pork, 10 tierces smoked ham, 14 tierces pickled ham, 100 barrels navy bread, 50 barrels pilot bread, 1,200 pounds green coffee, 900 pounds roasted coffee, 180 pounds tea, 120 gallons cider vinegar, 2 sacks salt, 30 pounds black pepper, 125 gallons molasses, 3 barrels hominy, 1,000 pounds navy tobacco, 500 pounds smoking tobacco, 4 pails fine cut tobacco, 20 boxes boneless codfish, 1 box desiccated codfish, 12 sacks dried apples, 3 dozen wash basins, 1 dozen lanterns, 240 coffee cups, 240 soup plates, 240 three-prong forks, 240 knives, 240 metal spoons, 10 coffee cans, 12 dishpans, 12 large tin dishes, 12 large tin buckets, 1 pair measuring scales, 1 coffee mill, 150 pounds adamantine candles, 480 pounds soap, 100 pounds salt water soap, 250 gum army blankets, 3 barrels granulated sugar, 2 barrels cut loaf sugar, 1 barrel fancy maple drip, 1 barrel extra syrup, 2 barrels New Orleans molasses, 15 barrels beans, 1 barrel plain pickles, 20 barrels onions, 100 barrels flour.

From a statement by Mr. Edward Atkinson it seems that the average cost of feeding a laboring man in 1884 was about 22 cents a day, and the cost of clothing was about 13 cents. The cost of the food is divided up as

follows: Meat, poultry and fish, 42 per cent.; milk, butter and cheese, 22; bread, 11; sugar, 9; tea and coffee, 5; vegetables, 4; fruit, 3; eggs, 2; salt, spices, ice, etc., 2 per cent. The Ohio Commissioner of Labor Statistics finds that the average cost of maintaining a mechanic in that State is 32 45 cents per day.

ADJUSTMENTS OF INSTRUMENTS.

The following is a convenient statement of the methods of adjusting the transit and level. It is taken from lectures delivered at the Rensselaer Polytechnic Institute, when the writer was a student there:

ADJUSTMENTS OF THE TRANSIT.

1st. To make the axes of the level tubes parallel to the graduated limb.

Test. Bring the bubbles to the centres of the tubes by the leveling screws, and turn the limb half-way around. The bubbles should remain at the centres.

Adjustment. Bring each bubble half-way back by the screws at the end of the tube.

In bringing the bubble to the centre of the tube by the leveling screws, it is well for the beginner to recollect that the opposite screws should be turned in pairs together, the thumbs approaching or receding from one another, and that the bubble will move in the direction of movement of the left thumb when it turns its screw.

2d. To make the line of collimation perpendicular to the axis of the trunnions.

Test a. Levèl the limb, and fix the cross-hairs on some well defined point on a level with the telescope and clamp.

b. Plunge the telescope and mark a point in the opposite direction now covered by the cross-hairs.

c. Unclamp and turn the limb until the cross-hairs cover the same point again, as at first, and clamp.

d. Plunge the telescope, and mark another point on a level with the telescope. This point should coincide with that marked in the process *b* above.

Adjustment. Move the vertical hair by the adjusting screws over one-fourth of the apparent distance from the last point marked, to the first. In making this adjustment, it is well to bear in mind that when the pins are inserted in the upper holes of the capstan-headed screws, pushing them from you makes the hair appear to move to the left.

3d. To make the axis of the trunnions parallel to the limb.

Test a. Level the limb carefully; fix the cross-hairs on some elevated point, and clamp.

b. Depress the telescope, and mark some low point covered by the cross-hairs.

c. Unclamp, and turn the limb half-way around; fix the cross-hairs on the elevated point and clamp.

d. Depress the telescope, and mark some low point covered by the cross-hairs ; this point should coincide with the former low point.

Adjustment. Lower the end of the axis opposite the second low point, by means of the adjusting screws on the standard.

4th. To centre the eye-piece. Although this is not necessary for accurate work, it is more agreeable to the eye.

Adjustment. Move the eye-piece by its adjusting screws until the intersection of the cross-hairs appears to be in the center of the field of view.

ADJUSTMENTS OF THE LEVEL.

1st. To make the line of collimation coincide with the axis of the telescope.

Test a. Fix the intersection of the cross-hairs on some well defined point by means of the leveling screws, and clamp.

b. Roll the telescope half over. The intersection should remain on the point.

Adjustment. Bring each cross hair half way back to the point by its adjusting screws.

2d. To bring the axis of the telescope and attached level into the same plane.

Test. Bring the bubble to the centre, and roll the telescope a little to one side and the other. The bubble should be stationary.

Adjustment. Bring the bubble back to the centre by the horizontal adjusting screws.

3d. To make the axis of the attached level parallel to the axis of the telescope.

Test. Bring the bubble to the centre and reverse the telescope end for end, in the Ys. The bubble should return to the centre.

Adjustment. Bring the bubble half-way back by the adjusting nuts at the end of the tube.

4th. To make the axis of the attached level perpendicular to the vertical axis.

Test. Bring the bubble to the centre over one pair of leveling screws, and reverse the bar. The bubble should remain at the centre.

Adjustment. Bring the bubble half-way back by the adjusting nuts at the end of the bar. In repeating the test, place the bar over the other pair of leveling screws.

5th. To centre the eye-piece. This is the same as for the transit.

6th. To make the object piece move parallel to the axis of the telescope.

Test a. *Adjust* the line of collimation on a *distant* object.

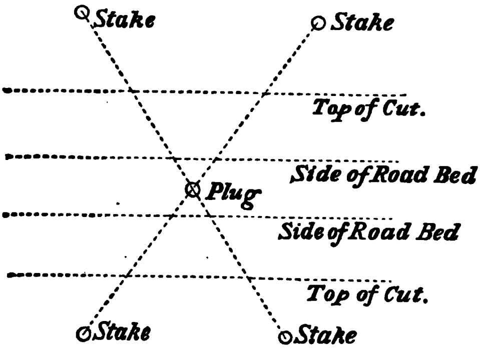
b. Fix the cross-hairs on a *very near* object, and roll the telescope half over. The intersection should remain on the point.

Adjustment. Bring each cross-hair half-way back by the adjusting screws of the object piece. In repeating the test, make use of the near object first.

The vernier of transits reads either to minutes or to hundredths of a degree, as the purchaser prefers. The latter method will be found much more convenient for running in curves. To find the least reading of any transit, let n equal the number of spaces on the vernier, corresponding to m spaces on the limb, and a equal a single space on the limb; then the least reading equals $\frac{a}{n}(n-m)$. Usually $n - m = 1$, and the least reading becomes $\frac{a}{n}$.

CONSTRUCTION.

The road is divided up into lengths of about thirty miles, each of which is placed in charge of a "Principal Assistant Engineer." Each of these divisions is subdivided into lengths of about seven miles, and given in charge of an Assistant Engineer, whose party consists of a Rodman, Chainman and Axeman. The first work of the Assistant should be to retrace the line and test the bench marks. All plugs should be guarded, and a bench should be made at every culvert. There are various modes of guarding plugs—by intersecting lines, by distances from other plugs, or a combination of the two. The best method, where the ground admits of it, is by intersecting lines.

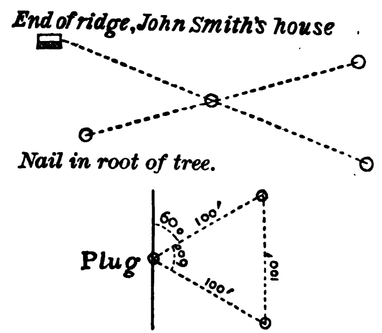


A part of a house, or the corner of a window or chimney, may often be substituted for one of the above stakes, for a foresight.

The note book may be kept in the form as on the following page.

Station	Plug	Distance	Deflection	Reading	Total Angle	Remarks
87	P. T.	86 + 92	55′	8°35′	11°30′	Straight.
86	......	....	1°00′	7°40′	....	....
85	......	....	1°00′	6°40′	....	....
84	Tr. P.	84	1°00′	2°50′	5°40′	....
83	......	....	1°00′	1°50′	....	....
82	......	....	0°50′	0°50′	....	....
81	P. C	81 + 17	....	....	....	2° curve.

Topography.



The advantage of this form of field-book is, that having first made and checked it in the office, there will be no more calculating of curves in the field, and so much less risk of error. It would always be necessary to run from the same end of the curve, and to use the same transit points; but this is no objection, as the plugs are all guarded, and it is as easy to set over one as another. Guarding all the plugs saves a great deal of trouble in re-running the line after grading, when it never measures the same as before, and it is difficult to run the old line without the *same* points to run from.

At the end of the transit note-book, a page should be devoted to each culvert, giving its station and a little plot of the stakes set, of course drawn roughly; and a little drawing of the culvert; also the level of the bridge seat and foundations, etc.

The staking out for excavation is done in several ways. In a tolerably flat or undulating country, it is generally done with the level; on steep hillsides, two rods are used, one ten feet long, and the other of any convenient length, divided into feet by different colors. That which is ten feet long is held horizontally by means of a hand-level laid upon it, with one end resting upon the ground, and the other against the shorter rod, which is held vertically. It is raised until it is horizontal, and the height read off the vertical rod by the Rodman, and noted by the Assistant on a piece of loose paper. He calculates where the slope runs out, and, having checked it on the ground, or made a closer approximation, enters it in a special field-book. The same principle governs the setting of the slope stakes with the level and level-rod. As a large part of the Assistant Engineer's work consists in setting slope-stakes, a more minute description is perhaps necessary. They are set opposite the centre line stakes, at the tops of the cuts and bottoms

of the fills. If the slope is $1\frac{1}{2}$ to 1 and the half width of the road-bed is called b, the horizontal distance of the slope-stake from the centre line is called x, and the height of the slope-stake above the sub-grade is called h; then

$$x = b + 1\tfrac{1}{2}\, h.$$

By assuming a value of x, measuring it out, and finding the corresponding value of h, with the level or the two rods, the values are substituted in the above equation: if both sides are the same, the assumed value of x was correct, and the stake should be driven in. If, however, the left-hand side is greater or less than the right-hand, the position of the stake should be moved toward or away from the centre line an estimated amount, and the process repeated of taking a new height with the level, and making a new calculation, until the two sides agree. After a little practice, it will not, usually, be necessary to make more than two trials. The following is the form of field-book used:

Station........	Ground.. ..	Grade	Width of road bed........	Slope.........	Cross Sections.					
270	114.6	109.3	32	1:1	$\frac{+6.2}{22.2}$	$\frac{+5.9}{16}$	+5.3	$\frac{+4.7}{16}$	$\frac{+4.6}{18}$	$\frac{+9.2}{25.2}$
+ 60	117.4	109.9	32	1:1	$\frac{+7.5}{23.5}$	$\frac{+7.5}{16}$	+7.5	$\frac{+9.8}{16}$		$\frac{+11.4}{27.4}$
271	118.0	110.3	32	1:1	$\frac{+8}{24}$	$\frac{+7.9}{16}$	+7.7	$\frac{+11.1}{16}$		$\frac{+14}{30}$
+ 13	116.0	110.4				$\frac{0}{16}$	+5.6	$\frac{+7.8}{16}$		$\frac{+11.9}{27.9}$
+ 54	110.9	110.9					0.0			
272	109.8	111.3	26	1½:1	$\frac{-2}{16}$	$\frac{-1.9}{13}$	−1.5	$\frac{0}{13}$		
273	107.2	112.3	26	1½:1	$\frac{-8}{25}$	$\frac{-6.6}{13}$	−5.1	$\frac{-4.8}{13}$		$\frac{-4.7}{20.1}$

It will be noticed that in these notes levels have been taken at the half-widths of the road-beds. This is to facilitate calculating the areas of the cross-sections.

Cross-sections are also taken at the points where the edges of the road-bed go from cut to fill, to facilitate calculating the quantities. The portions between these cross-sections and where the edges run from cut to fill are usually pyramids, to be calculated by the rule of the base by one-third the height, and are to be added to the amounts calculated between the other cross-sections by the prismoidal formula. For the case of the ground being level at right angles to the line at the passage from cut to fill, these additional solids are wedges, whose contents are to be calculated by the prismoidal formula as in the portion between full stations. Some engineers look upon the calculation of the point where the slope runs out, in the field, as a waste of time; and only take the transverse slope, being sure to take it far enough out. They then plot the cross-section, and take the distance to the slope-stake from the plot with a scale. They claim that this method is advantageous, too, because they always run out further than necessary for the slope, and if, afterward, as often happens, the slope will not stand, but slides out—a "slip"—they still have a record of the amount which slides by measuring to the top of the slide, while, too often, when such an accident occurs, the Assistant finds that he has no note of the slope of the ground beyond his stake, which has been carried away. When such an event occurs, it is better not to slope the cut further up, but to take away the earth at the level of the road-bed for some distance in, to catch any further slide before reaching the track, although the slope may be steeper than was intended.

In staking out with the level, it is well to have a number of sheets of paper, fastened together at the edges, for

making trial calculations on; when one is covered with figures, it can be torn off and thrown away, exposing another. The cross-sections should be plotted in a permanent record book, to be kept in the office. The area of each should be calculated. For applying the prismoidal formula for calculating the cubic contents, it is requisite to know the middle cross-section between each two that are measured on the ground. The closest approximation to this is the following: A new cross-section is drawn, in which each linear dimension is a mean of the corresponding linear dimensions of the end sections; the area is then calculated. When the end sections are not of the same number of sides, the one with the least number should have a sufficient number of points assumed on the straight sides to make the number the same, by regarding a straight side as being formed of two adjacent sides. The areas are then substituted in the formula:

$$S = \frac{l}{6} (A + 4M + A')$$

where A and A' are end areas and M is the middle area, and l is the distance of end stations apart.

A method sometimes pursued for obtaining the middle area is to transform the end areas into figures of equal area, but with horizontal ground surfaces, calculate the "equivalent centre depths" or the depth from the horizontal surface to the point where the side slopes meet, and take a mean centre depth for the middle section, calculate its corresponding area, substitute in the formula and finally substract the "grade triangle" or the constant quantity included between the slopes produced and the road-bed. Tables have been constructed of "equivalent centre depths" for various areas, and other tables give the cubic contents at once, for a given depth and given slopes, from the equivalent centre depths of the end sections.

Professor Rankine recommends a different method of finding the middle cross-section. Instead of finding the "equivalent centre depths" of the end-sections, and taking the mean for the centre depth of the middle-section, he assumes that it has a depth equal to the mean of the two actual centre-depths of the end-sections, and a slope equal to the harmonic mean of the slopes at the ends; from these data the middle section is calculated. In other words, if a and c are the slopes at the ends, the slope of the middle section is taken as $\frac{2\,a\,c}{a+c}$. If the slope of the ground changes in a cross-section, it is not clear how this method can be applied. When the cross-sections are measured at equal distances, as is generally the case in ground with easy slopes, it being sufficient to take them only at each even chain's length, the labor of calculation may be very much abridged by using the following formulas, also from Rankine:

When there is an even number of equidistant cross-sections:

Let A, A', A'',. . . . .A^m be the successive cross-sections and l their distance apart; then the total volume is

$$S = l \left\{ \frac{A}{2} + A' + A'' + \&c. \ldots\ldots + \frac{A^m}{2} \right\}.$$

When there is an odd number of equidistant cross-sections $A, A', A'', \ldots\ldots A^n$

$$S = \frac{l}{3} \left\{ A + 4\,A' + 2\,A'' + 4\,A''' + 2\,A''' + \&c. \ldots\ldots + 2\,A^{n-2} + 4\,A^{n-1} + A^n \right\}.$$

It will be noticed that the last, which is Simpson's rule, is the calculation by the prismoidal formula, regarding the successive prismoids as two stations long, while the first is

the calculation by mean areas. That by Simpson's rule being the most accurate, it will be better to leave off one station when there is an even number and use the second formula, afterwards calculating the amount in the omitted station and adding it. Even this method, however, supposes that the ground is a prismoid between these stations, with the cross-section measured at the middle one what it would actually be if the supposition were correct. Practically, on ordinary ground, this is the case, though it is always the most correct method to form the middle section as first described by taking mean measurements from the end sections and calculating each prismoid between stations by itself. However, it should be attempted in the field notes, to take the cross-sections sufficiently often to make the volumes between successive ones true prismoids, when the calculated mean section would agree with the actual one.

When the cross-sections are bounded, at the top in cuts, or at the bottom in fills, only by lines joining the centre-stake to the side-stakes, a convenient form of the prismoidal formula for calculating, at once from the field notes of staking out, is as follows:

$$S = \frac{l}{6}\,\frac{(2\,H + H')\,(d_1 + d_1') + (2\,H' + H)\,(d_2 + d_2')}{2} - T\,l$$

in which H and H' are the respective centre cuts or fills at the ends, plus the height of the triangle formed by the side slopes produced, with the road-bed; d_1, d_1'; d_2 and d_2' are the side distances, and T is the area of the above-mentioned triangle formed by the side-slopes, prolonged, with the road-bed. For the same width of road-bed and same side-slopes, the number to be added to the end centre-cuts or fills to obtain H and H' will be constant, as will be likewise T. This supposes the greater value of the centre cut or fill and the greater value of the sum of the side distances

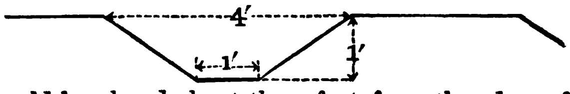
to occur on the same cross-section, as they usually do. If, however, these greater values occur at different ends, it will only be necessary to transpose H and H' in the formula. The following is another form of the same equation, which may perhaps be preferred. It is given by Prof. Greene in *Engineering News*, Oct. 2, 1880 :

$$S = \frac{l}{6}\left\{(c_1 + \tfrac{1}{2}c_2)(d_1 + d_1') + (c_2 + \tfrac{1}{2}c_1)(d_2 + d_2') + \tfrac{1}{2}w\,(h_1 + h_1' + h_2 + h_2')\right\},$$

where c_1 and c_2 are the centre-cuts or fills, d_1, d_1', d_2 and d_2' are the side distances, w is the width of the road-bed, and h_1, h_1', h_2 and h_2' are the side heights.

Many engineers complain of the great labor of the calculations involved in using these formulas, although, in construction, there is usually ample time on rainy days to perform them, when there may be nothing else to do in the office. To save this labor, Mr. A. M. Wellington has lately published a work in which the quantities may be taken from graphical tables, to which the reader is referred.

At the top of cuts it is well to have a ditch made on the up-hill side to keep the slope from being washed down. Proper dimensions are :



It should be placed about three feet from the edge of the slope.

It will sometimes be found that a cut passes through ground filled with springs, which come out on the sides of the slopes, washing them down, and continually filling up the ditches and increasing largely the amount of material to be removed. In such cases it is better to dig ditches in the face of the slope at right-angles to the centre-line, and

fill them with broken stone, so as to form drains for the water, leading it to the side ditches. They may be made two feet wide and four feet deep, and about twenty feet apart, this distance however, of course varying with the amount of water delivered.

Estimates of the work done are taken up each month. It is important that all papers containing notes of the measurements should be preserved. Although these estimates are only intended as rough approximations, the measurements taken will often prove of service in following estimates.

CULVERTS.

For finding the proper water-way to give to culverts, the drainage area of the stream should be discovered if possible. Where county maps are obtainable, this can easily be measured from them. If the drainage area is small, it may often be estimated by walking round it. The water-way may then be calculated by the following formula of Mr. E. T. D. Myers :

$$A = c\ \sqrt[4]{M},$$

in which A is the area of the opening of the culvert in square feet, M is the drainage area in acres, and c is a variable co-efficient, depending on the country, and for which Mr. Myers recommends $1\frac{6}{10}$ in hilly, compact ground, and 1 in comparatively flat ground. In mountainous, rocky country, this value may often be raised to 4, and in very exceptional cases as high a value as 10 has been observed, a case being reported in Mississippi (Transactions Am. Soc. of C. E., vol. 25, p. 116) where, in Rocky Creek, a value of c would be required of 48. Inquiry should be made of the neighboring people to learn the greatest height of floods in the stream, and the vertical dimensions of the water-way may be made equal to the flood height of the stream at the spot, although this is not necessary.

BOX CULVERTS.

Rule for laying out on the ground : Take the height of the top of the parapet from the height of the embankment at the centre, and with the remainder (considered as height of embankment) find the side distances with the level as in setting slope-stakes ; then add 18 inches at each end, and if the height of the embankment exceeds 10 feet, add one inch on each end for every foot in height above the parapet.

The covering flags are one foot thick and the parapet one foot high, making two feet from top of abutments to top of parapet. For the thickness of abutments take ⅓ the height of embankment on top of abutments, observing, however, that the abutments must never be less than two feet nor more than four feet thick. To determine the length of the wings, add the height of the opening to the thickness of the flags ; one and a half times this sum, added to two feet, will give the distance from the end of opening to end of wing ; the wing to be at right-angles to the drain, unless the latter be askew ; then the wings to be parallel to the direction of the railroad. Instead of digging deep foundations, the method now employed is to put in a paving made of stones a foot deep, set up on edge, with a curb two feet deep at each face of the drain, and to start the walls on this paving. Should the fall of the drain not exceed 9 inches, make the pavement level, dropping the upper end 9 inches below the surface. Should the fall be greater, make a sufficient number of drops of 9 inches each in the length of the drain. At every drop place a cross-sill 2 feet deep ; the wings and parapet to be of the same thickness as the abutments. The above rule was adopted on the construction of the Junction Railroad, of Philadelphia. Some engineers prefer, instead of building wings at right angles to the culvert, to continue the side walls past the faces, and slope them down with the

embankment to its foot by steps. When driftwood is liable to stop up the waterway, it is well to carry the upper end up plumb without steps, so that the water may run over the drift and fall into the culvert.

The vertical dimension of the water-way in a box culvert is usually 6 inches or a foot larger than the horizontal one. Some engineers think that $2\frac{1}{2}$ by 3 feet is the smallest that should be allowed, that being considered the smallest size that will admit of a man entering to clean it in case it becomes stopped up. Others think a 2 by $2\frac{1}{2}$ is enough to admit a boy. Most build the end walls on foundations of their own, only the body of the culvert being on a pavement, when the span is greater than $2\frac{1}{2}$ feet. Ordinary sizes for box culverts are $2\frac{1}{2}$ by 3, $2\frac{1}{2}$ by $3\frac{1}{2}$, 3 by 4, and sometimes, when good stone is plenty, 4 by 5. The latter would require a thicker covering stone than 1 foot; it should be 15 inches. When the span is only $2\frac{1}{2}$ feet, a covering stone 10 inches thick will be sufficient.

There should be a foot of earth over the covering stones to act as a cushion to the moving load. When the available distance for the height of the culvert is not sufficient to obtain the requisite water-way, two or more openings are made, alongside of each other, separated by a 2-feet wall, each of the same relative dimensions as a single opening.

When the pavement is on soil which is easily washed out, a layer of broken stone should be placed under it at the ends. A velocity of $2\frac{1}{2}$ feet per second is sufficient to cut away a channel in ordinary soil containing more sand than clay.

OPEN CULVERTS.

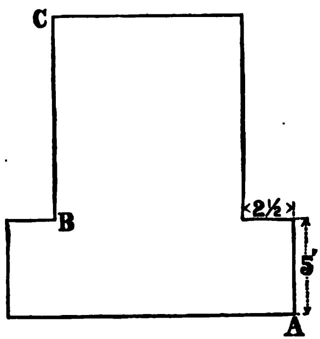
These are generally made of two feet span, with walls two feet thick, with a depth of not more than three feet, founded on a paving, one foot thick. The ends slope down at the same rate as the bank, or $1\frac{1}{2}$ to 1.

CATTLE GUARDS.

These are often placed on each side of a public road crossing, when this takes place at grade. They are built like open culverts, with spans varying from three to five feet, and about three feet deep. Stringers 12" × 12" placed 3 feet 11 inches in the clear, support the rails, of a sufficient length to rest well on the solid wall on each side. Two struts, 5" × 12", and 4 feet 6 inches long, and mortised into each stringer 3½ inches, are placed about six inches further apart than the span of the opening, and the stringers are held to them by a rod one inch diameter, 6 feet 5 inches long, with square nuts and long flat washers, placed by each strut. As cattle sometimes get wedged in these pits and derail a train, a series of pointed slats is often used instead, over which it is difficult for the animals to walk, but which will not admit a falling through into a pit. These are not so effective, however, as the pits for restraining the passage of the animals.

OPEN PASSAGE-WAYS.

These are made either with wing-walls or "T" abutments. When with wing-walls, the thickness at the base



should be calculated like a retaining wall ($\frac{3}{7}$ the height).

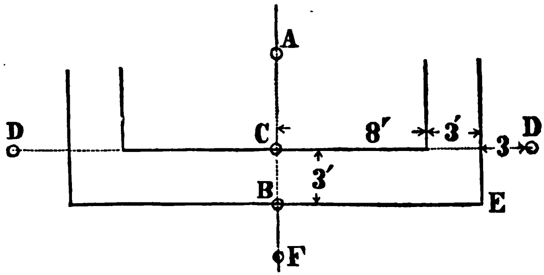
The wing-walls are usually placed at an angle of 45° with the centre line, that angle requiring least masonry. The coping then slopes down at the rate of 2.12 to 1, which is a good proportion for steps, if they are preferred. If the road is for a single track, the "T" abutment will be found more economical. The length of the "T" should be so calculated that the earth sloping down it at the rate of $1\frac{1}{2}$ to 1, and striking the back of the bridge-set, and then one end, should just strike the ground at the corner of the bridge-seat, or as near it as the Engineer desires. For instance, suppose the distance from A to sub-grade is 12 feet; then

$$12 \times 1\tfrac{1}{2} = 5 + 2\tfrac{1}{2} + x, \text{ or } x = 10\tfrac{1}{2} = \text{length of } B\,C.$$

In staking out for a passage-way, always make the pit a foot larger all round than the foundation is intended to be, so that the quality of the masonry can be seen. The mason would prefer to fill up the entire pit. If the passage-way is on a curve, having decided where one face should come, turn off from the nearest plug the angle corresponding to the sub-chord to the face, and put in a plug; set up over this, and turn off the sub-chord to the face of the other abutment. (Deflection for a sub-chord in minutes is equal to $\frac{3}{10}$ × degree of curve × length of sub-chord in feet.) Turn off right-angles from this last sub-chord at each of these plugs, and put in others outside of the pits for the mason to stretch his line by, for the faces of the abutments. Plugs should also be put on both sides of the last sub-chord produced, beyond the pits, to give the centre line of the bridge. This finishes the instrumental work, the other stakes being put in with a tape. A convenient way of doing this is shown on next page.

Let $A\,F$ be the centre line, marked with plugs at A and C, and let $C\,D$ be the face of the neat work. A stake is to be put in at the corner of the pit E, the pit being sup-

posed to be 3 feet larger all around than the neat work. Lining by eye, put in a stake B 3 feet from C. Then, with the ring of a tape at B and the 17-feet mark at D, take hold of the 14-feet mark and draw the tape tight; the 11-feet mark will give the point E.



It is well to give the mason a sketch on a piece of paper, giving all dimensions, drawn on the spot by eye without scale, and let him do his own marking out on the foundation. The pit is a sufficient guide for putting in the foundation. After setting the stakes for the pit, take levels at each one and note in the book; also note the depth of the pit before the masonry is begun, so that the cubic contents can be calculated. A level has also to be taken at the face before laying out the neat work, to give the height of the neat work to bridge-seat and for calculating the batter and span at the bottom. A 12-feet span bridge, 12 feet high, with a batter of one-half an inch to the foot, would be only 11 feet span at the base of the neat work.

STONE ARCHES.

Rankine's rule for the depth of the keystone in feet:

For a single arch, $D = \sqrt{.12\,R}$.
For an arch in a series, $D = \sqrt{.17\,R}$.

in which R is the radius at the crown in feet,

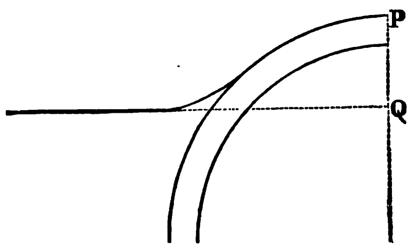
This is for circular or segmental arches. For elliptical arches, for R substitute $\frac{a^2}{b}$ when the earth is dry, or $\frac{4a^2}{b}$ when the earth is wet, a being the half-span, and b being the rise.

Trautwine's rule is :

$$D = \frac{\sqrt{R + \frac{1}{2} S}}{4} + .2 \text{ foot},$$

in which R is the radius of the circle which will touch the crown and the springing lines, and S is the span.

Rankine gives as the thickness of the abutment from $\frac{1}{3}$ to $\frac{1}{5}$ of the radius at the crown (for abutment piers, $\frac{1}{2}$ the radius), and for the thickness of the piers $\frac{1}{6}$ to $\frac{1}{7}$ of the span. He says to make the masonry of the pier solid up to the point where a line from the centre of the arch to the extrados forms an angle of 45° with the vertical. Fill in the backing before striking the centres to such a height that $P\,Q = \sqrt{r'r - r^2}$, where r is the radius of the intrados, and r' is the radius of the extrados.



Trautwine gives for the thickness of the abutment at the springing line, when the height above the ground of this line is not more than $1\frac{1}{2}$ times the base,

$$= \frac{\textit{Rad. in ft.}}{5} + \frac{\textit{rise in ft.}}{10} + 2 \text{ feet},$$

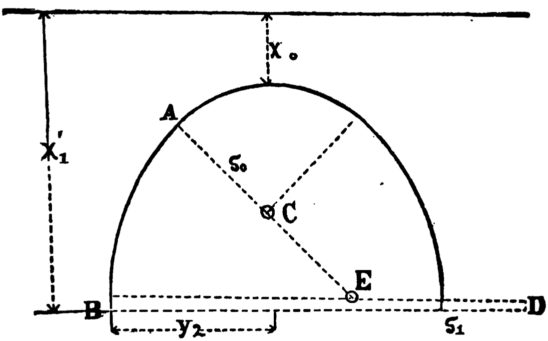
He batters the back of the abutment at the rate of one inch per foot, and carries it above the springing line to a height of one-half the distance from the springing line to the top of the arch, and slopes it from this point on a line tangent to the arch.

A common way of proportioning the water-way is to make the height of wall forming the abutment, where the height of the embankment permits it, one-half the span of arch, when the latter is full centred. Ordinary sizes of arches are 6, 8, 10, 12, 14, 17, and 20 feet span.

The wing walls may be at right angles to the face of the arch, but are generally placed at an angle of 45 or 60 degrees with the centre line. They are sometimes battered three inches to a foot on the face, the back being plumb. If the latter makes an angle of 60 degrees with the centre line, it can easily be calculated that the neat work on the face starts from the top of the foundation with an angle of 52 degrees with the centre line, in order to have the wall of the same width, say 2 feet, at the top, where it intersects the 1½ to 1 slope. The thickness of such a wall is not at a horizontal section equal to three-sevenths the height as it should be as a retaining wall, but is found sufficient, receiving some support from the culvert at its end. It may be stopped at any height at the end that may be desired, so long as the slope round its end does not reach too near the water course to be washed away. The height is usually made from 3 to 5 feet at the ends.

If the embankment over the arch is very high, or if the arch is the lining to a tunnel in earth, the proper form for the intrados is a geostatic arch. Rankine's approximate formula for this is:

$$y_2 = \tfrac{1}{3}(x_1 - x_0)\sqrt[3]{\frac{x_1}{x_0}}\left\{1 - \frac{1}{30}\sqrt[3]{\frac{x_1}{x_0}}\right\}$$



In any given case x_1 and y_2 will be known, and we can calculate x_0 by trial ; $x_1 - x_0$ will then be the rise of the arch, which we shall call a. The geostatic arch will approach a five-centre curve, which may be drawn as follows :

$$\text{Calculate } b = \tfrac{1}{2}\, y_2 + \frac{\frac{1}{4}\, y_2^2}{30\, a}.$$

$$\text{Then } \varsigma_0 = A\,C = \frac{a}{2}\left(1 + \frac{b^2}{a^2}\right) \text{ and } \varsigma_1 = B\,D = \frac{a}{2}\left(1 + \frac{a^2}{b^2}\right).$$

From these equations we obtain the centres C and D. About D with a radius $D\,E = \varsigma_1 - a$ describe a circular arc, and about C with a radius $C\,E = a - \varsigma_0$ describe another arc ; the intersection of these at E will be the third centre. For convenience in calculating x_0 the following table may be used :

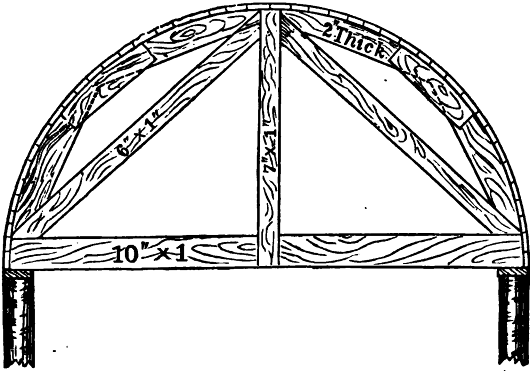
Values of x_0 when

$x_1 =$	$y_2 = 5$	$y_2 = 10$	$y_2 = 15$	$y_2 = 20$
10	3	.8	.2	.1
15	7	3	1.1	.5
20	11.3	6	3	1.5
25	16	9.6	5.7	3.3
30	20.8	13.9	9	5.7
35	25.6	18.3	12.6	8.6
40	30.5	22.8	16.5	11.9
45	35.4	27.3	20.7	15.5
50	40.3	31.8	25	19.2

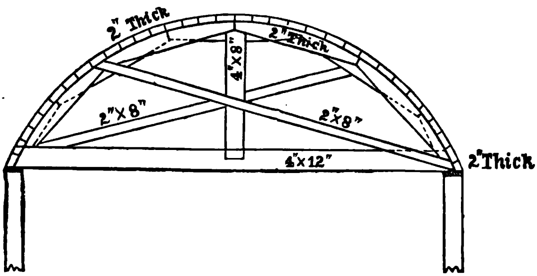
When brick is plentiful, circular culverts are often employed. They require no foundation except for the face

walls. For the thickness, one brick (nine inches) is sufficient for a span less than six feet. Add one more brick for each six feet more of span up to thirty feet.

The following are two cheap forms of centring:



14-feet span. Frames 2½ feet apart.



24 feet span, 8-feet rise. Frames 3½ feet apart.

Post mortised (by slight tenon) into chord. Arch pieces pinned together and halved in chord and post; braces spiked at ends; and at intersection with post a ½-inch bolt is used.

Centres had better remain under the arch as long as possible. In stone arches the parapet should not be made too high, or it may be pushed over by the bank; it is well to proportion it like a retaining wall if more than

one or two courses high. Some loose stones laid flatwise behind it will relieve the thrust of the earth.

In designing centres, allow $\frac{1}{300}$ of the span for settling of the arch, unless built very slowly and with great care.

RETAINING WALLS.

Let b = the breadth at the bottom = 1.
" h = the height.
" t = the thickness at the bottom.
" w = the weight of a unit of volume of masonry.
" w' = the weight of a unit of volume of earth.
" θ = the slope of the bank above the wall.
" φ = the angle of repose of earth.
" j = the inclination of the foundation pit to the horizon.
" q = a constant of safety

$$= \frac{\text{distance from middle of base to point where the line of resistance cuts base}}{t}$$

It is always between 0 and $\frac{1}{2}$.

Let q'

$$= \frac{\text{distance from the middle point of base to point where base is cut by a vertical line through the centre of gravity}}{\text{thickness at base.}}$$

It is always less than $\frac{1}{6}$.

$$\text{Let } n = \frac{\text{total weight of masonry}}{w\,h\,b\,t}$$

$$\text{Let } w_1 = w' \cos.\theta \,\frac{\cos.\theta - \sqrt{\cos.^2\theta - \cos.^2\varphi}}{\cos.\theta + \sqrt{\cos.^2\theta - \cos.^2\varphi}}.$$

Rankine then gives :

$$\frac{t}{h} = \sqrt{\frac{w_1 \cos.\theta}{6\,n\,(q \pm q)\,w \cos.^2 j} + \left(\frac{w_1\,(q + \frac{1}{2})\sin.(\theta + j)}{4\,n\,(q \pm q')\,w \cos.^2 j}\right)^2} - \frac{w_1\,(q + \frac{1}{2})\sin.(\theta + j)}{4\,n\,(q \pm q')\,w \cos.^2 j.}$$

$$\text{If } \theta = 0;\ w_1 = w' \frac{1 - \sin.\ \varphi}{1 + \sin.\ \varphi.}$$

$$\text{If } \varphi = 35^\circ;\ w_1 = .27\ w'.$$

If $n = \frac{1}{2}$ and $j = 0$ in addition,

$$\frac{t}{h} = \sqrt{\frac{.27\ w'}{3\,(q \pm q')\,w.}}$$

If we suppose the wall to be just stable without the least excess of strength, $q = \frac{1}{2}$. It is customary, however, to give q a smaller value, so that there will be an excess of strength; otherwise the pressure would be concentrated at a point, and it would split off or crush. The English engineers make $q = .375$.

For q' we can assume the following values for walls of different heights:

Height of wall.	q'
20	.11
35	.14
55	.15
100	.16

For first-class masonry we may take $\frac{w'}{w} = \frac{100}{165}$

For dry sand-stone rubble we may take " $= \frac{100}{120}$

Then if the wall is over 100 feet high, of first-class masonry,					$\frac{t}{h} =$	.51
"	"	"	"	dry rubble	"	.60
"	" between 55 and 100 ft. high, first-class				"	.49
"	"	"	"	dry rubble	"	.58
If the wall is between 35 and 55 feet high, of first-class					"	.48
"	"	"	"	" dry rubble	"	.56
"	" less than 20		"	first-class	"	.45
"	"	"	"	dry rubble	"	.53

When the wall is rectangular in section:

If of first-class masonry, $\frac{t}{h} = .27$

" dry rubble " " .32

When $\theta = \varphi$, all the previous values of $\frac{t}{h}$ become more according to Rankine's formula.

M. Boussinesq multiplies the value of w_1 on the preceding page by $\frac{\cos.(\frac{1}{4}\pi - \frac{1}{2}\varphi)}{\cos.(\frac{3}{2}\varphi - \frac{1}{4}\pi)}\cos.\varphi$. He likewise says that a closer value will be obtained if, instead of φ we write $\varphi' = \frac{\varphi + \varphi''}{2}$, in which φ'' is found from the equation

$$\sin.\varphi'' = \frac{\sin.\varphi + \sqrt{8 + \sin.^2\varphi}}{4}\sin.\varphi.$$

With the value of $\varphi = 35°$, φ'' will equal $29° 44'$, and φ' will equal $32° 22'$. This will give the value of $w_1 = .123\, w,'$ and if the lower part of the double sign is used, which corresponds with ordinary walls, we have

$$\frac{t}{h} = \sqrt{\frac{.123\, w'}{3\,(q - q')\, w}}$$

This will give values of $\frac{t}{h}$ of about one-third less than those on the previous page, except for the rectangular section.

The following are the rules used by different authorities :

In a discussion before the American Society of Civil Engineers ("Transactions," vol. 3, p. 75), a Canadian engineer was quoted as giving $\frac{t}{h} = \frac{3}{8}$ for first-class masonry laid in hydraulic cement. A rule used on the Pennsylvania Railroad is $\frac{t}{h} = \frac{3}{7}$. Rankine gives as the ordinary English rule, $\frac{t}{h} = .41$, and for a very safe rule $\frac{t}{h} = .48$.

Trautwine gives :

For rectangular walls of first-class masonry,	$\frac{t}{h}$	=	.35
and of mortar rubble or brick	"	"	.40
and of dry rubble	"	"	.50

When the walls are offset at the back, he recommends a thickness at the base of about $\frac{1}{2}$ more, and at the top of $\frac{1}{2}$ less, containing the same amount of masonry. (See his book.)

It was the practice on the Pennsylvania Railroad to make the base $\frac{3}{7}$ of the height, and after carrying the back plumb for three or four feet to make a step, calculating the new thickness of wall at $\frac{3}{7}$ of the remaining height, and so continuing to step off to the top, where a thickness of three feet was given.

In railroads along a river bank, where the embankment slopes into the river, the slopes are "pitched" with stone about two feet long, laid upon the slope, at right-angles to it. They should start at the bottom in a trench dug about three feet below the surface of the ground.

TUNNELS.

A "heading" is first driven. This is about five or six feet high, and as wide as the nature of the material will allow. In earth it may only be three feet, while in solid rock it should be of the full width of the tunnel. In earth, it is generally driven at the bottom of the section of the tunnel; in this case, chambers are often excavated of the full size at intervals in the heading, and the work prosecuted from each in both directions until they meet. In solid rock, the heading should be at the top of the section of the tunnel, unless very wet, and the enlargement should be carried on as closely to the heading as possible, say within fifty feet, although where machine-drills are used, it may not be possible to keep so close. After the heading a "bench" is made about 7 feet above grade of the full width of the tunnel, and about 30 feet back from the heading, and finally the bottom is taken out the full width

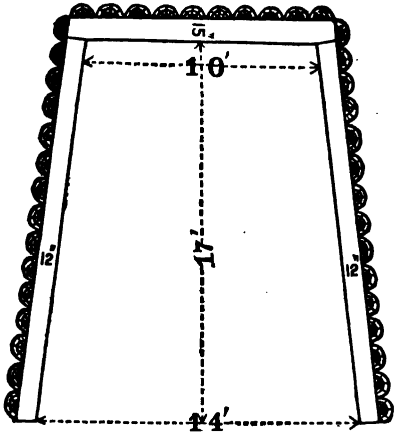
of the tunnel. With machine drills the tendency seems now to be to put the heading at the bottom always, in rock as well as in earth, especially in very long tunnels.

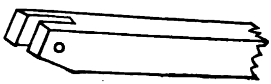
When the two ends of the tunnel are so near the same level that the difference in their heights is not sufficient to secure a proper fall for drainage, a summit should be made in the tunnel. On the Mont Cenis tunnel a fall of .05 per 100 was considered sufficient, but Mr. B. H. Latrobe recommended .5 per 100. The St. Gothard has .1 per 100 (although level for 1,500 feet in the middle); Musconetcong, .15 per 100.

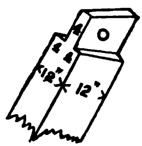
When shafts are sunk for the purpose of accelerating the work by having so many additional faces to work from, they should be filled up on the completion of the tunnel, as they interfere with the ventilation. It is found that the ventilation of the Hoosac tunnel is very bad since completion. It has a shaft in the middle which is left open, and acts well in removing the smoke from one end, but not from the other; the clear end depending, it is believed, on the direction of the wind.

When the tunnel is through earth or rotten rock, it will require to be arched. This is done with either brick or stone; in the London clay, which swells on being exposed to the air, it required a thickness of 54 inches of brickwork to withstand the pressure; 18 inches to two feet is the ordinary thickness. The Metropolitan Railroad, in London, goes under warehouses eighty feet high with a thickness of fourteen bricks, laid in cement, with a layer of concrete on top.

The proper form for the intrados of a tunnel through sand, or some such substance which acts only by its weight, is the geostatic arch, to which a near approximation is made, when the load is infinite, in an ellipse with the semi-vertical axis double the semi-horizontal, or the rise equal to the span. In substances like the London clay, however,







which swell on exposure to the air, a circular form is probably the best for the intrados. In soft material, the heading and the enlargement have to be timbered as they advance. The above was the method adopted on the Northwestern Virginia Railroad :

Legs squared 12″ and 17½′ long. Cap, 12 feet long, 15 × 12 inches. Lagging half round, split out, about six inches thick, long enough to lie on two bents. After it was put in, the earth was rammed into the intervening space.

For the timbering on the Central Pacific Railroad, a longi-

tudinal sill on each side, 12″ × 12″, carried the posts, 12″ × 16″, inclining outward at top, at intervals of 1½ to 5 feet. On these posts arches were made (polygons of seven sides) of three thicknesses of 5″ × 12″ plank, bolted with ¾ inch bolts. Width of sub-grade inside posts was 17 feet, and at springing line 19 feet. Height of crown above grade, 19 feet 9 inches. Split lagging on top, 2½ inches thick.

The timbering should be put in large enough for the masonry to be built inside; the earth is then tamped in above, the timbering sometimes remaining in, although it had better be taken out.

For running the line in rock tunnels, the transit points are made in the heading by driving wooden plugs in the roof, and centering them.

The excavation for the St. Gothard tunnel is 8 meters wide by 6 meters high, exclusive of space for the masonry. The heading was 2.4 meters high and 2.6 meters wide, kept about 200 or 250 meters ahead of the enlargement, at the top of the enlarged section. The enlargement is first made by cutting the place for the roof. About 200 or 250 meters further back, a cutting is made about 3 meters wide, down to the floor of the tunnel. About the same distance still further back, the whole section is excavated, and the remaining masonry put in. The heading of Clifton tunnel was 8 × 10 feet.

In the heading for a tunnel on the Great Western and Midland Railroads, at Bristol, thirty to forty shots were required to bring away the face, the holes being three feet six inches deep. They were exploded successively, beginning with the central holes, which were angled, and progressing to the outside ones. At the Mont Cenis, the machines were too long to allow of putting the first holes at an angle, and the first opening was made by putting down larger holes in the centre, which were not fired.

At the St. Gothard tunnel, the three central holes formed a triangle sixteen inches apart, converging to four inches at the bottom at a depth of 3½ to 4 feet.

In the Musconetcong tunnel, a slope was made, instead of a shaft, 8×20 feet in the clear, at an angle of 30 degrees. Through earth it was timbered with collars 12×12 inches oak, 4 feet apart, centre to centre, supported by end and two middle props, lagged at the sides and above with chestnut "forepoling." Through rock the dimensions were 8×16 feet. Top headings were started in the tunnel 8×26 feet wide. Where the rock was disintegrated, collars of 15 inches oak, set 5 feet apart, were used, lagged above and sometimes at the sides, and supported either on legs or by hitches in the rock. These collars were sufficiently high to clear a two-foot ring of masonry, and about them packing was securely blocked in, up to the roof. The heading at the end of the tunnel was made 26 feet wide by 7 feet high. A heading through earth was made 8 feet at top and 10 feet at bottom, and 8 feet high, with oak collars and props of 12 to 15 inches round timber; sets placed 2½ to 2 feet apart, centre to centre, footed in very soft ground on six-inch sills, but ordinarily on three-inch foot-blocks. This information is obtained from Mr. Drinker's "Tunnelling," p. 221.

Tunnels are usually made 22 feet high from sub-grade to top, and 16 feet wide for single track and 26 feet for double track. Single track tunnels generally have vertical sides and a semi-circular top; and those for a double track have a cross-section composed of arcs of different circles tangent to each other, the upper half approximating a semi-circle whose centre is about nine feet above sub-grade. For various sections, and an immense amount of information on tunnelling, see Mr. H. S. Drinker's book. A common way of "bonding" the brick in tunnel linings is, to

lay two consecutive courses of stretchers until the outer course falls behind the inner one just the thickness of a brick ; the interval in the outer course is then filled in with stretchers, and a heading course follows. In a semicircular arch of 16 feet span, these heading courses will occur at intervals of about 21 bricks. If the span is 26 feet, the heading courses would occur every 35th. As this is scarcely often enough, it is better to lay alternate courses of headers and stretchers, making the surfaces of the headers radial by thickening the mortar joint at the outer end. The successive nine-inch rings thus formed should be tied together with headers whenever their joints come in line, which will be about every 17th course.

It may be interesting to note that when the headings met in the St. Gothard Railroad tunnel, 14,920 meters long, the alignment was out 33 centimeters laterally, five centimeters vertically, and seven meters in length. (See *Engineering News* for June 5th, 1880).

In obtaining the centre line in deep shafts, points are established at the top on each side from which plumb lines can be suspended. To prevent the oscillations of the bob, where the line is long, it is sometimes placed in water, oil, or tar. In a shaft 450 feet deep, in Wales, where the points on the line at the top of the shaft were only ten feet apart, Mr. McGregor used plumb bobs weighing ten pounds each, and passed the line just above the bob through a slot ($1\frac{1}{2}$ by $\frac{1}{4}$ inch wide) at one end of a needle $9\frac{3}{4}$ inches long, balanced on a pivot like a compass needle, but with one end twice as long as the other, so as to magnify the oscillations. These needles faced each other, and rested on boards on which the amplitude of the oscillations could be marked. By bisecting these amplitudes and prolonging the lines, if each struck the other pivot, the latter was in the centre line required. If either did not strike this point, the board was moved over an es-

timated amount, and the experiment repeated until the pivots and the bisections of the amplitudes were in the same line. (Proceedings Institution of Civil Engineers, Vol. 73, part 3.)

The depth of a shaft is measured with wooden rods having hooks and eyes on the opposite ends, one linking into another. The length of each is carefully measured between the points of contact, and the sum will, of course, give the total length.

BRIDGES.

Simple beam uniformly loaded, rectangular :

Let $W =$ the breaking weight in pounds.

" $b =$ the breadth of the beam in inches

" $d =$ the depth of the beam in inches.

" $L =$ the length of the beam in inches.

" $S =$ a constant which has been determined by experiment.

The value of S is, for oak, 10,000 ; for white pine, 7,000 ; for wrought iron, 40,000 ; for cast iron, 30,000.

$$W = \frac{4}{3} S \frac{b\, d^2}{L.}$$

Beam uniformly loaded, cylindrical :

Let $r =$ the radius in inches, the other letters being as before

$$W = \frac{2\,S}{L} 3.1416\, r^3.$$

Beam uniformly loaded, I-shaped section :

If the flanges are of the same size, as they always are in rolled beams :

Let $d =$ the depth of one of the flanges.

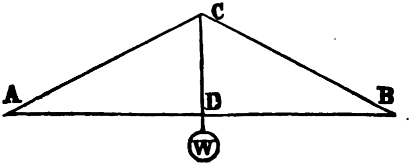
" $d' =$ the depth of the connecting piece.

" $A =$ the area of one of the flanges.

" $A' =$ the area of the connecting piece.

$$W = \frac{4}{3} \frac{S}{L} \left\{ 4\,(d + d')\,A + \frac{d'^2\,(A + A')}{d' + 2\,d} \right\}.$$

When the load is supported in the middle, instead of being uniformly distributed, the breaking load in each of the foregoing cases becomes only half as much.

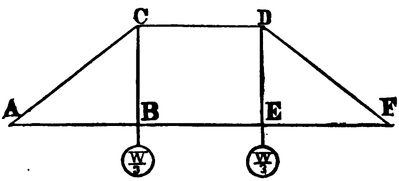


The king-post truss :

$$\text{Stress on } C\,D = W; \text{ stress on } A\,C \text{ and } C\,B = \frac{W}{2}\,\frac{A\,C}{C\,D.}$$

$$\text{Stress on } A\,B = \frac{W}{2}\,\frac{A\,D}{C\,D.}$$

If the load is uniformly distributed, it will produce the same effect as one-half the above load suspended in the middle. The beam is supposed to have no stiffness at D. Actually, $A\,B$ is always made in one stick, and its stiffness will reduce the above values by an indeterminate amount



The queen-post truss :

$A\,B = B\,E = E\,F.$

Stress on $C\,B$ and $D\,E = \frac{1}{3}\,W.$

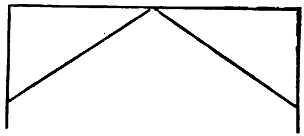
$$\text{Stress on } A\,C \text{ and } D\,F = \tfrac{1}{3}\,W \times \frac{A\,C}{C\,B}$$

$$\text{Stress on } C\,D = \tfrac{1}{3}\,W \times \frac{A\,B}{C\,B} = \text{stress on } A\,F.$$

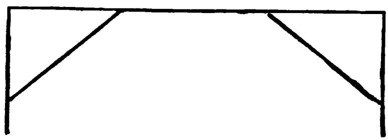
These are the stresses when loads of $\frac{1}{3}$ W are placed at B and E, or a total uniform load of W. In the latter case, the abutment at A has to sustain, in addition, the load on $\frac{1}{2}$ $A\,B$, which, added to the resolved component of the stress on $A\ C$, produces a vertical stress of $\frac{1}{2}$ W, as it ought.

If only the point B is loaded with $\frac{1}{3}$ W, the portion which is transferred to the abutment F will produce a moment about D tending to break the joint across, if it is rigid. If, however, it is flexible, there will be a tendency

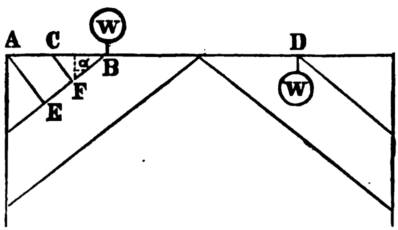
for the joint to rise which is resisted by the rod $D\ E$ (producing a stress upon it of $\frac{1}{8}\ W$) and the stress transferred to E, must be resisted by the transverse strength of the beam $B\ F$, calculated in the same way as the first case of the simple beam loaded in the middle (of a length $B\ F$, not $A\ F$). If braces are introduced in the directions $B\ D$ and $C\ E$, the bridge becomes a Howe truss.



Stresses are the same in this case as in the king-post truss, the tension on the straining beam, however, being converted into thrust against the abutments.



Stresses are the same as in the queen-post truss.

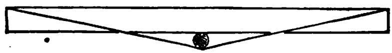


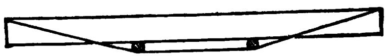
This is a combination of the king and queen post systems.

To prevent the point B from rising on the application of a weight at D, braces are often introduced at $A\ E$ and $C\ F$. The force W, acting upward at B, produces a force equal to W sin. α in the direction of $C\ F$ ($C\ F$ being at right angles to $F\ B$) and acting at B, which has a tend-

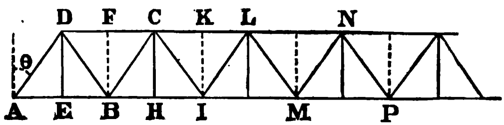
ency to break the beam transversely at F. The formula for the breaking weight of a rectangular solid beam fixed at one end and loaded at the other is $W = \frac{1}{6}\, S \frac{b\, d^2}{L}$. The pressure on $C\, F$ is also W sin. a, which produces a transverse stress in $A\ \ B$ at C, which can be calculated in like manner.

The trussed girders of the following forms may be looked upon as king and queen post trusses inverted:





They are objectionable as being a combination of two systems, and it is impossible to tell just how much of the load will be borne by the beam acting as such, or by the rods transmitting a portion of the stress to compress the beam. It depends on the adjustment of the rods.



The "Warren girder," or "triangular truss:"

Let W = the weight on one panel = the weight on $F C$, due to uniform load or weight of bridge.

" W' = the weight on one panel = the weight on $F C$, due to the variable load.

" n = the number of panels, $A\, E$, $E\, B$, etc.

If the load is on the upper chord, struts $F\, B$, $K\, I$, etc., are introduced, and if on the lower chord, rods $D\, E$, $C\, H$, etc., so that each apex of the triangles will bear a load. The maximum stress on one of these struts or ties would be $W + W'$.

The maximum compression on $A\ D = \frac{n-1}{2}(W + W')\ sec.\ \theta.$

The maximum compression on $D\ B = \frac{1}{n}\ W'\ sec.\ \theta - \frac{n-3}{2}\ W\ sec.$
θ, if a plus quantity; if it is a minus quantity, there will be no compression on $D\ B$.

The maximum tension on $D\ B = \frac{n-3}{2}\ W\ sec.\ \theta + \frac{(n-2)\ (n-1)}{2\ n}\ W'$
$sec.\ \theta.$

The maximum compression on $B\ C = \frac{n-5}{2}\ W\ sec.\ \theta + \frac{(n-3)\ (n-2)}{2\ n}$
$W'\ sec.\ \theta.$

The maximum tension on $B\ C = \frac{3}{n}\ W'\ sec.\ \theta - \frac{n-5}{2}\ W\ sec.\ \theta$, if a
plus quantity; if it is a minus quantity, there will be no tension on $B\ C$.

The maximum compression on $C\ I = \frac{6}{n}\ W'\ sec.\ \theta - \frac{n-7}{2}\ W\ sec.$
θ, if a plus quantity; if it is a minus quantity, there will be no compression on $C\ I$.

The maximum tension on $C\ I = \frac{n-7}{2}\ W\ sec.\ \theta + \frac{(n-4)\ (n-3)}{2\ n}$
$W'\ sec.\ \theta.$

The maximum compression on $I L = \frac{n-9}{2} W\ sec.\ \theta + \frac{(n-5)\ (n-4)}{2\ n}$
$W'\ sec.\ \theta.$

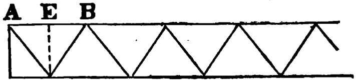
The maximum tension on $I\ L = \frac{10}{n}\ W'\ sec.\ \theta - \frac{n-9}{2}\ W\ sec.\ \theta$, if
a plus quantity; if it is a minus quantity, there will be no tension on $I\ L$.; etc., etc., to the middle of the bridge, when the order will be reversed.

The stresses on the chords are greatest when the bridge is uniformly loaded with its greatest load. We then have:

Tension on $A\ B = \frac{n-1}{2}(W + W')\ tan.\ \theta.$

Tension on $B\,I =$ tension on $A\,B + (n-4)\,(W + W')\,tan.\,\theta$.
Tension on $I\,M =$ tension on $B\,I + (n-8)\,(W + W')\,tan.\,\theta$.
Tension on $M\,P =$ tension on $I\,M + (n-12)\,(W + W')\,tan.\,\theta$,
etc., etc., etc.
Compression on $D\,C = (n-2)\,(W + W')\,tan.\,\theta$.
Compression on $C\,L =$ compression on $D\,C + (n-6)\,(W + W')\,tan.\,\theta$.
Compression on $L\,N =$ compression on $C\,L + (n-10)\,(W + W')\,tan.\,\theta$, etc., etc., etc.

If the roadway is on the upper chord, the inclined piece, *A D*, in the last figure, is sometimes left out, and the bridge built as follows:

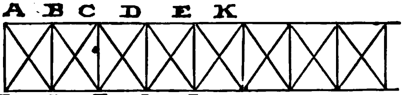


A D, *D B*, *D E*, etc., sustain the same amount of stress as before, but what was then compression is now tension, and *vice versa*. *A B* has the same amount of stress as *A B* in the other figure, and *D C* as *D C* the kinds being reversed as before. The part of the lower chord, *D O*, might, in this case, be dispensed with, were it not that it is of use in the lateral bracing, and must, therefore, be introduced.

The triangular truss is often built with two or more sets of different triangles forming the bracing; when of more than two, it is called a lattice truss. The stresses on these can easily be represented in formulas as above, but are omitted for the sake of brevity. Each separate system of triangles has stresses like the previous form, but independent of each other.

The Howe truss:

Let $n =$ the number of panels.
" $W =$ the weight on one panel due to the uniform load.
" $W' =$ the weight on one panel due to the variable load.



Maximum compression on $FB = \frac{n-1}{2}(W+W')\frac{FB}{BG}$

Maximum compression on $G\,C$

$$=\left\{\frac{n-3}{2}W+\frac{(n-2)(n-1)}{2n}W'\right\}\frac{FB}{BG}$$

Maximum compresssion on $H\,D$

$$=\left\{\frac{n-5}{2}W+\frac{(n-3)(n-2)}{2n}W'\right\}\frac{FB}{BG}$$

Maximum compression on $I\,E$

$$=\left\{\frac{n-7}{2}W+\frac{(n-4)(n-3)}{2n}W'\right\}\frac{FB}{BG}$$

etc., etc. By continuing this process past the middle of the bridge, we obtain the maximumstresseson the counter-braces, and when a stress becomes minus, it shows that beyond that point the counter-braces may be left out.

Maximum stress on $B\,C$

$$=\frac{n-1}{2}(W+W')\frac{FG}{BG}$$

= maximum stress on $F\,G$.

Maximum stress on $C\,D$

$$=\left\{\frac{(n-1)+(n-3)}{2}(W+W')\right\}\frac{FG}{BG}$$

= maximum stress on $G\,H$.

Maximum stress on $D\,E$

$$=\frac{(n-1)+(n-3)+(n-5)}{2}(W+W')\frac{FG}{BG}$$

= maximum stress on $H\,I$.

Maximum stress on $E\,K$

$$=\frac{(n-1)+(n-3)+(n-5)+(n-7)}{2}(W+W')\frac{FG}{BG}$$

= maximum stress on $I\,L$.

These calculations should be stopped at the middle panel; the other side will be the same

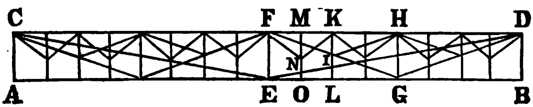
The maximum stress on $B\ G = \frac{n-1}{2}(W + W')$ for a through bridge, or W' less for a deck bridge.

The maximum stress on $C\ H = \frac{n-3}{2} W + \frac{(n-2)(n-1)}{2n} W$ for a through bridge, or W' less for a deck bridge.

The maximum stress on $D\ I = \frac{n-5}{2} W + \frac{(n-3)(n-2)}{2n} W$ for a through bridge, or W' less for a deck bridge.

The Pratt truss has the same outline as the Howe truss, but the verticals are struts, and the diagonals ties. The stresses would be the same as those for the Howe truss, except that $F\ G$, $G\ H$, etc., should be changed to $A\ B$, $B\ C$, etc., and $F\ B$, $G\ C$, etc., to $A\ G$, $B\ H$, etc. In the Pratt truss, however, the stresses on the verticals in a through bridge are as above for the Howe deck bridge, and W' *more* for the deck bridge.

Both trusses are sometimes made "double intersection," and the remarks on the Warren truss will apply also to these.



The Fink truss:

The through bridge, supposed to have sixteen panels.

Every part will receive its maximum stress when the bridge is uniformly loaded.

Let S = the span $A\ B$.
" D = the depth of truss $F\ E$.
" L = length of main suspending rod $C\ E$.
" L' = length of suspending rod $F\ G$.
" L'' = length of suspending rod $H\ I$.
" L''' = length of suspending rod $F\ N$.
" W = total weight of bridge and load.

The stress on center post $F\ E$ and end post $D\ B = \frac{W}{2}$

The stress on quarter post $H\ G = \frac{W}{4}$

The stress on eighth post $K\ I = \frac{W}{8}$

The stress on sixteenth post $M\ N$ = weight of $\frac{1}{4}$ chord $F\ K$ and $\frac{1}{4}$ of lateral braces.

The stress on suspending links $I\ L$ and $N\ O$ = weight of one panel with load.

The stress on suspending rod $E\ D = \frac{W}{4}\ \frac{L}{D}$

The stress on suspending rods $D\ G$ and $F\ G = \frac{W}{8}\ \frac{L'}{D}$

The stress on suspending rods $H\ I$ and $F\ I = \frac{W}{16}\ \frac{L''}{\frac{1}{2}D} = \frac{W}{8}\ \frac{L''}{D}$

The stress on suspending rods $K\ N$ and $F\ N = \frac{W}{32}\ \frac{L'''}{\frac{1}{2}D} = \frac{W}{16}\ \frac{L'''}{D}$

The stress on the chord is found by resolving the stresses which are transmitted by the tension rods. Then

Stress on chord

$$= \frac{W}{4}\ \frac{\frac{1}{2}S}{D} + \frac{W}{8}\ \frac{\frac{1}{4}S}{D} + \frac{W}{16}\ \frac{\frac{1}{8}S}{\frac{1}{2}D} + \frac{W}{32}\ \frac{\frac{1}{16}S}{\frac{1}{2}D} = \frac{45}{256}\ \frac{WS}{D}$$

The deck bridge : In this case, the suspending links $N\ O$ and $I\ L$, etc., will be left out. The stress on the chord and tension rods and all the posts except the "sixteenth," $M\ N$, etc., will be the same as before. The sixteenth posts, $M\ N$, etc., will be strained by an amount equal to $\frac{W}{16}$

For bridges up to 20 feet span rolled I-beams are generally used. From 20 feet to 60 feet, plate iron bridges, and for greater spans trusses are built. For plate-iron bridges a stress sheet is prepared as though it were a Pratt truss, the stiffening pieces which divide it into panels being regarded as posts and the plate as the tension braces.

The thickness of the plate is then calculated such that the section at right angles to a diagonal, less the section of the rivet holes, shall safely resist the maximum stress, viz., that on an end panel; and this thickness of plate is generally continued throughout the bridge. The chords and posts should be calculated to withstand their direct stresses obtained from the stress-sheet, and in addition the chords should be regarded as uniformly loaded beams between panel points, the stresses in the web being so transferred to them; placing them in the condition of uniformly loaded beams with a thrust (or pull in the lower chord) in the direction of their length. These stresses transferred from the web to the chords, however, are not those on the stress-sheet, since the maximum stress on the plate in any panel will not occur at the same time as the maximum stress on the chord. We must use instead only the first term of the stresses given in the diagonal on the stress-sheet, and for W substituting $W + W'$.

ON THE MOST ECONOMICAL HEIGHT AND STYLE OF BRIDGE TRUSS.

(*Partly from Proceedings of the Engineers' Club, of Philadelphia.*)

In many cases of bridge design, the height of the truss is fixed by some extraneous local condition, which does not belong to the bridge itself; such as the available head room, either between the trusses or under them, or a desire to limit the area exposed to the wind, etc. In the great majority of cases, however, the engineer has a great latitude in deciding on the ratio of the height of the truss to the span. This ratio is generally chosen by copying some existing bridge, and an arbitrary one-eighth or one-

tenth the span is chosen, without considering very thoroughly what the proper proportion should be. For a given span, a given style of truss, a given number of panels, and a given load, there is a certain height of truss which will require least material, and is therefore the cheapest. To discover what this particular height should be, is not a very difficult problem. The ratio of height to the length of a panel is expressed by a letter m, and the stresses on the various parts written, being functions of m. The proper sections being found from the stresses are multiplied by the lengths of the various pieces, and then by the price per cubic unit of the material, and added together. The value of m corresponding to the least value of this function is found by the principle of minima in the calculus, making the first differential coefficient equal to zero, etc.

To apply this to the Howe truss:

Let n = the number of panels.
l = the length of a panel.
W = the weight on one panel due to the uniform load.
W' = the weight on one panel due to the variable load.
h = the least dimension of a brace.
h' = the least dimension of the upper chord.
p = the cost in dollars, per 1,000 ft., board measure, of lumber.
p' = the cost in cents, per pound, of the iron.
m = the height of truss divided by the length of one panel.
1,000 lbs. = the safe load on the wood per square inch.
10,000 lbs. = the safe load on the iron per square inch, and let Gordon's formula be used for calculating the strengths of the chord and braces, each being proportioned, in each panel, to its safe load.

We then have for the total cost of one-half truss:

$$
\left[
\begin{array}{l}
\dfrac{1}{1000}\left\{
\begin{array}{l}
(n-1)(W+W') + (n-3)W + \\
\dfrac{(n-2)\ (n-1)}{n} W' + (n-5)W \\
+ \dfrac{(n-3)\ (n-2)}{n} W' + (n-7) \\
W + \dfrac{(n-4)\ (n-3)}{n} W' + \text{\&c.,} \\
\text{until the algebraic sum of} \\
\text{these last two terms is a mi-} \\
\text{nus quantity.}
\end{array}
\right\} l\left(1+\dfrac{l^2}{250\,h^2}\right) \\
+\dfrac{1}{1000}\left\{
\begin{array}{l}
[n-1] + [(n-1) + \\
(n-3)] + [(n-1) + \\
(n-3) + (n-5)] + \\
\text{\&c., until the last} \\
\text{term in the brack-} \\
\text{ets } [\ldots.] \text{ is } 3.
\end{array}
\right\} (W+W')\ l\left(1+\dfrac{l^2}{250\,h'^2}\right) \\
+\dfrac{1}{2000}\left\{
\begin{array}{l}
[n-1] + [(n-1) + (n-3)] + \\
[(n-1) + (n-3) + (n-5)] + \\
\text{\&c.,} \\
\text{until the last term in the} \\
\text{brackets } [\ldots.] \text{ is } 1.
\end{array}
\right\} (W+W')\ l
\end{array}
\right] \times \frac{p}{1440} \times \frac{1}{m}
$$

$$
+\left[
\begin{array}{l}
+\dfrac{1}{1000}\left\{
\begin{array}{l}
(n-1)(W+W') + (n-3).\ W \\
+\dfrac{(n-2)\ (n-1)}{n} W' + (n-5)\ W + \\
\dfrac{(n-3)\ (n-2)}{n} W' + \text{\&c.,} \\
\text{until the algebraic sum of these} \\
\text{last two terms is a minus quantity.}
\end{array}
\right\} l\left(1+\dfrac{2\,l^2}{250\,h^2}\right)\dfrac{p}{1440} \\
+\dfrac{1}{20000}\left\{
\begin{array}{l}
(n-1)(W+W') + (n-3)W + \dfrac{(n-2)\ (n-1)}{n} \\
W' + (n-5)\ W + \dfrac{(n-3)\ (n-2)}{n} W' + \\
\text{\&c.,} \\
\text{until the last term in the numerator} \\
\text{is } 3.
\end{array}
\right\} l\,\dfrac{28\ p'}{100}
\end{array}
\right] \times m
$$

$$+\frac{1}{1000}\left\{\begin{array}{l}(n-1)\,(W+W')+(n-3)\;W+\\ \dfrac{(n-2)\,(n-1)}{n}\,W'+(n-5)\;W+\\ \dfrac{(n-3)\,(n-2)}{n}\,W'+\text{\&c.,}\\ \text{until the algebraic sum of these last}\\ \text{two terms is a minus quantity.}\end{array}\right\} l\,\frac{l^2}{250\,h^2}\,\frac{p}{1440}\times m^2$$

This may be written in the form :

$$v=\frac{a}{m}+b\,m+c\,m^2$$

Putting its first differential coefficient equal to zero, we obtain the value of

$$m=\sqrt{\frac{\sqrt{12\,a\,c+b^2}-b}{6\,c}}$$

We can now apply these equations to some practical examples.

The variable load may be taken, for a railroad bridge at 1½ tons per lineal foot, which gives in pounds $2\,W'=3360\,\frac{l}{12}$ or $W'=140\,l.$

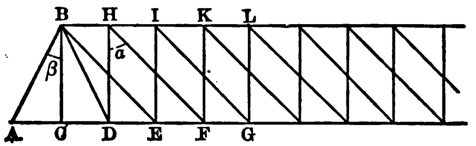
By using the empirical formula of a subsequent page for calculating W, reducing to pounds and inches.

$$W=\frac{280}{3}\left\{\frac{2+\frac{n\,l}{192}}{17+\sqrt{\frac{12}{n\,l}}}\right\}l$$

If $n=12$, $l=120$, $h=8$ and $h'=11$, $p=27$, $p'=5$, we have $W=6{,}216$, $W'=16{,}800$, $m=.96$ and cost of one-half truss $=$ \$336.78. If $n=6$, $l=249$, $W=12{,}432$ and $W'=33{,}600$, the other quantities being as before, we have $m=.87$, and cost of one-half truss=\$353.17. If, in the last case, however, we make $h=12$ and $h'=20$, we have $m=.73$ and cost of one-half truss=\$304.94, which shows the result of using

pieces of large diameter in the compression parts. It would seem, then, that when only small scantlings can be obtained, it would be better to make a large number of panels, and when large scantlings are procurable, to increase the length of the panels. It should be observed, however, that the load is supposed to be applied to the truss only at the panel points. As a matter of fact, it is usually applied on a chord, and the additional material required in the chord on this account would be proportionately greater in a long panel than in a shorter one. It is better, however, to use separate pieces to support the load between the panel points, and so confine the chords to their legitimate duty of resisting the stresses in the direction of their lengths due to the structure acting as a truss, only.

In order to find the values of a, b and c for a double intersection Pratt truss bridge, the following statement of the maximum stresses on the several parts will be found of use:



Maximum compression on $BH = \left(\frac{3n-4}{4} \tan. \beta + \frac{n-4}{4} \tan. \alpha\right)$ $(W + W')$

Maximum compression on HI = max. comp. on $BH + \frac{n-6}{4}$ $(W + W') \tan. \alpha$.

Maximum compression on IK = max. comp. on $HI + \frac{n-8}{4}$ $(W + W') \tan. \alpha$.

Maximum compression on KL = max. comp. on $IK + \frac{n-10}{4}$ $(W + W') \tan. \alpha$.

&c. = &c.

Maximum tension on $A\,D = \frac{n-1}{2}(W + W')\,tan.\,\beta.$

Maximum tension on $D\,E =$ max. ten. on $A\,D + \frac{n-2}{4}(W + W')$

$tan.\,\beta.$

Maximum tension on $E\,F =$ max. ten. on $DE + \frac{n-4}{4}(W + W')$

$tan.\,\alpha.$

Maximum tension on $F\,G =$ max. ten. on $E\,F + \frac{n-6}{4}(W + W')$

$tan.\,\alpha.$

&c. = &c.

Maximum compression on $A\,B = \frac{n-1}{2}(W + W')\,sec.\,\beta.$

Maximum tension on $B\,C = \frac{1}{2}W + W'.$

When n is even:

Maximum tension on $B\,D = \left(\frac{n-2}{4}W + \frac{n-2}{4}W'\right)sec.\,\beta.$

Maximum tension on $B\,E = \left(\frac{n-4}{4}W + \frac{(n-2)^2}{4\,n}W'\right)sec.\,\alpha.$

Maximum tension on $H\,F = \left(\frac{n-6}{4}W + \frac{(n-4)\,(n-2)}{4\,n}W'\right)$

$sec.\,\alpha.$

Maximum tension on $I\,G = \left(\frac{n-8}{4}W + \frac{(n-4)^2}{4\,n}W'\right)sec.\,\alpha.$

&c. = &c.

Maximum compression on $H\,D = \frac{n-4}{4}W + \frac{(n-4)\,(n-2)}{4\,n}W'.$

Maximum compression on $I\,E = \frac{n-6}{4}W + \frac{(n-4)^2}{4\,n}W'.$

Maximum compression on $K\,F = \frac{n-8}{4}W + \frac{(n-6)(n-4)}{4\,n}W.$

&c. = &c.

When n is odd:

Maximum tension on $B\,D = \left(\frac{n-2}{4}W + \frac{(n-1)^2}{4\,n}W'\right)sec.\,\beta.$

Maximum tension on $B\,E = \left(\frac{n-4}{4}W + \frac{(n-3)(n-1)}{4n}W'\right)$ $sec.\ \alpha$.

Maximum tension on $H\,F = \left(\frac{n-6}{4}W + \frac{(n-3)^2}{4n}W'\right) sec.\ \alpha$,

Maximum tension on $I\ G = \left(\frac{n-8}{4}W + \frac{(n-5)(n-3)}{4n}W'\right)$ $sec.\ \alpha$.

&c. = &c.

Maximum compression on $H\,D = \frac{n-4}{4}W + \frac{(n-3)^2}{4n}W'$.

Maximum compression on $I\ E = \frac{n-6}{4}W + \frac{(n-5)(n-3)}{4n}W'$.

Maximum compression on $K\,F = \frac{n-8}{4}W + \frac{(n-5)^2}{4n}W'$.

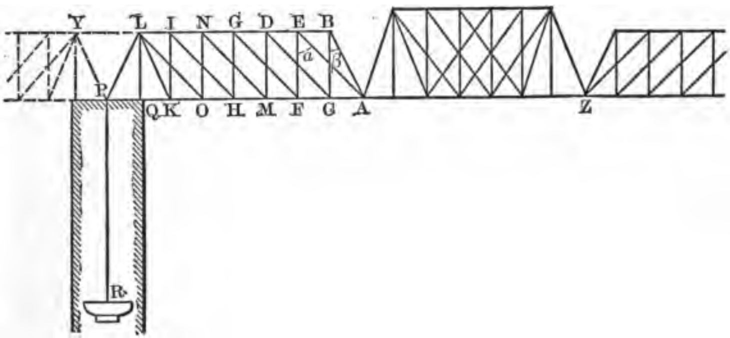
Maximum compression on $L\,G = \frac{n-10}{4}W + \frac{(n-7)(n-5)}{4n}W'$.

&c. = &c.

If, after calculating the most economical height for a single intersection bridge, the ties are found to approach more nearly to a vertical than 45°, it will be better to change the design to one of a double intersection, calculate its economical height and cubic contents, and compare with the first, to see if less material is not required. When the span approaches 500 feet, it will be found that the inclination of the ties for the most economical height in a double intersection bridge will make a less angle with the vertical than 45°, and then it will be better to try a treble intersection. An engineer who wishes to make a specialty of bridge designing can of course calculate in advance, in this way, at what span it is best to change from a single to a double intersection, and likewise where to change to a treble, etc., intersection. This supposes that a constant length of panel has been found to be the best for an economical superstructure. This seems to be

about 25 feet in the long span railroad bridges at present. The tendency, however, is to use only single intersection, and longer panels, if necessary.

Where, owing to the swiftness of the current, or some other cause, it is difficult to erect false works, it may be advantageous to erect a cantilever bridge. The following statement of the stresses in such a bridge may be useful:



The middle truss is supposed to be jointed with a pin at either A or Z, and to rest on rollers at the other end, so that the cantilevers shall be free to deflect without bringing any additional compression on the lower chords. The part $P\ Q$ must be made to resist compression, either by a continuation of the lower chord or by very carefully constructed masonry. If there is another span adjoining, a great deal of material in $P\ L$ and $P\ R$ can be saved by connecting the trusses by links $Y\ L$, so that their weights balance over the pier. $L\ Q$ may be made a masonry pillar and a saving of iron effected.

Let $W =$ the uniform load on one panel.
" $W' =$ the variable load on one panel.
" $W_2 =$ the weight of one-half of truss $A\ Z$ with its load.

Then

Maximum tension on $A\ B = \frac{1}{2}(W + W')\ sec.\ \beta + \frac{1}{2}\ W_2\ sec.\ \beta.$
Maximum tension on $D\ C = \frac{}{2}(W + W')\ sec.\ \alpha + \frac{1}{2}\ W_2\ sec.\ \alpha.$
Maximum tension on $N M = \frac{5}{2}(W + W')\ sec.\ \alpha + \frac{1}{2}\ W_2\ sec.\ \alpha.$
Maximum tension on $L\ O = \frac{7}{2}(W + W')\ sec.\ \alpha + \frac{1}{2}\ W_2\ sec.\ \alpha.$
Maximum tension on $A\ E = \frac{1}{2}(W + W')\ sec.\ \alpha + \frac{1}{2}\ W_2\ sec.\ \alpha.$
Maximum tension on $G\ F = \frac{3}{2}(W + W')\ sec.\ \alpha + \frac{1}{2}\ W_2\ sec.\ \alpha.$
Maximum tension on $I\ H = \frac{5}{2}(W + W')\ sec.\ \alpha + \frac{1}{2}\ W_2\ sec.\ \alpha.$
Maximum tension on $L\ K = \frac{7}{2}(W + W')\ sec.\ \beta + \frac{1}{2}\ W'_2\ sec.\ \beta.$
Maximum tension on $P\ L = 7\,(4\,W + 4\,W' + W_2)\,\frac{1}{cos.\ \beta}.$
Maximum tension on $E\ B = \frac{1}{2}(W + W' + W_2)\ tan.\ \beta.$
Maximum tension on $D\ E = \frac{1}{2}(W + W' + W_2)\,(tan.\ \alpha + tan.\ \beta)$
Maximum tension on $G\ D = 2\,(W + W')\ tan.\ \alpha + \frac{1}{2}(W + W')$
$tan.\ \beta + W_2\ tan.\ \alpha + \frac{1}{2}\ W_2\ tan.\ \beta.$
Maximum tension on $N\ G = \frac{7}{2}(W + W')\ tan.\ \alpha + \frac{1}{2}(W + W')$
$tan.\ \beta + \frac{3}{2}\ W_2\ tan.\ \alpha + \frac{1}{2}\ W_2\ tan.\ \beta.$
Maximum tension on $I\ N = 6\,(W + W')\ tan.\ \alpha + \frac{1}{2}(W + W')$
$tan.\ \beta + 2\,W_2\ tan.\ \alpha + \frac{1}{2}\ W_2\ tan.\ \beta.$
Maximum tension on $L\ I = \frac{17}{2}(W + W')\ tan.\ \alpha + \frac{1}{2}(W + W')$
$tan.\ \beta + \frac{5}{2}\ W_2\ tan.\ \alpha + \frac{1}{2}\ W_2\ tan.\ \beta.$
Maximum compression on $A\ C = (W + W' + W_2)(tan.\alpha + tan.\beta).$
Maximum compression on $F\ C = 2\,(W + W')\ tan.\ \alpha + \frac{1}{2}(W +$
$W')\ tan.\ \beta + W_2\ tan.\ \alpha + \frac{1}{2}\ W_2\ tan.\ \beta.$
Maximum compression on $M\ F = \frac{7}{2}(W + W')\ tan.\ \alpha + \frac{1}{2}(W +$
$W')\ tan.\ \beta + \frac{3}{2}\ W_2\ tan.\ \alpha + \frac{1}{2}\ W_2\ tan.\ \beta.$
Maximum compression on $H\ M = 6\,(W + W')\ tan.\ \alpha + \frac{1}{2}(W +$
$W')\ tan.\ \beta + 2\ W_2\ tan.\ \alpha + \frac{1}{2}\ W_2\ tan.\ \beta.$
Maximum compression on $O\ H = \frac{17}{2}(W + W')\ tan.\ \alpha + \frac{1}{2}(W +$
$W')\ tan.\ \beta + \frac{5}{2}\ W_2\ tan.\ \alpha + \frac{1}{2}\ W_2\ tan.\ \beta.$
Maximum compression on $K\ O = 12\,(W + W')\ tan.\ \alpha + \frac{1}{2}(W +$
$W')\ tan.\ \beta + 3\ W_2\ tan.\ \alpha + \frac{1}{2}\ W_2\ tan.\ \beta.$
Maximum compression on $Q\ K = 12\,(W + W')\ tan.\ \alpha + 4\,(W +$
$W')\ tan.\ \beta + 3\,W_2\ tan.\ \alpha + W_2\ tan.\ \beta.$
Maximum compression on $P\ Q$ = maximum compression on $Q K$.
Maximum compression on $B\ C = \frac{1}{2}(W + W' + W_2).$
Maximum compression on $D\ M = \frac{3}{2}(W + W') + \frac{1}{2}\ W_2.$
Maximum compression on $N\ O = \frac{5}{2}(W + W') + \frac{1}{2}\ W_2.$
Maximum compression on $E\ F = \frac{1}{2}(W + W') + \frac{1}{2}\ W_2.$
Maximum compression on $G\ H = \frac{3}{2}(W + W') + \frac{1}{2}\ W_2.$

Maximum compression on $I\ K = \frac{5}{2}(W + W') + \frac{1}{2} W_2$.
Maximum compression on $L\ Q = 11\ W + 11\ W' + 2\ W_2 + (4\ W + 4\ W' + W_2)(3\ tan.\ \alpha\ cot.\ \beta)$.
Maximum tension on $P\ R = 7\,(4\ W + 4\ W' + W_2)$.

Let $tan.\ \beta = \frac{1}{m}$; $tan.\ \alpha = \frac{2}{m}$; r = the least radius of gyration of the section of the chord; r_1 = the least radius of gyration of a post; 10,000 pounds = the safe load in tension; Rankine's formula for columns be used with a factor of safety of $\frac{1}{5}$. The total cubic inches in one-half truss $= \frac{A\ C}{108,000,000}(62\ W + 62\ W' + 19\ W_2)$ $\left(57,600 + \frac{A\ C^2}{r^2}\right)\frac{1}{m} + \frac{11\ A\ C}{3,750}(4\ W + 4\ W' + W_2)\,m + \frac{11\ A\ C}{216,000,000}$ $(4\ W + 4\ W' + W_2)\,m^3 + \frac{7\,(4\ W + 4\ W' + W_2)}{10,000} \times$ by length of $P\ R$ in inches.

The length of $P\ R$ of course depends on the weight of masonry above the bottom plate necessary to resist the upward pull on it. If there is another similar truss to the left of P, weighing as much as the truss $A\ P$ plus half the weight of $A\ Z$, and they are connected together by the links $L\ Y$, the latter will bear a maximum stress of $7\ tan.\ \beta\ (4\ W + 4\ W' + W_2)$. The maximum stress on $P\ L$ will then be relieved by $\frac{7}{cos.\ \beta}(3\ tan.\ \alpha + tan.\ \beta)$ $(4\ W + W_3)$, in which W_3 is the uniform load on one-half the intermediate truss $A\ Z$. The stresses on $P\ R$ and $L\ Q$ will then be proportionately reduced. One objection to this style of truss is that very wide piers are required, and the adjustment between the parts cannot be sufficiently accurately made as to make the stresses perfectly determinate. If the span on the left is anchored down at the end, $L\ Q$ will be entirely relieved of compression, and the width of the pier can be reduced. In such a case, however, its chords would have to be made to resist both tension and compression. The stresses in this case will become perfectly determinate.

For the part to the left of P, the stresses, when unloaded, would be found like those in $A\ P$, using the proper value of W, leaving out W' and using for W_2 a value equal to

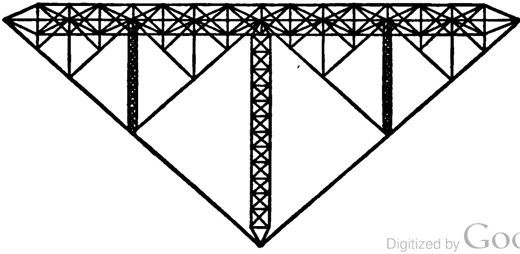
$$\frac{\text{stress on } L\ Y \times L\ Q}{\text{length of span to left of } P\ A}$$

$$\text{or } W_2 = \frac{7\ tan.\ \beta\,(4\ W + 4\ W' + W_2)\ B\ C}{A\ P}$$

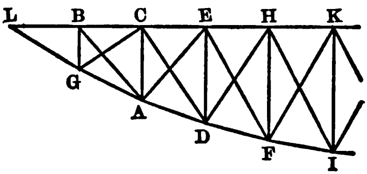
These should be combined with those produced in the span by the loads, considering it as a Pratt truss, to obtain the maximum stresses under any condition of loading.

It is the opinion of the author, as is further remarked on page 104, that the most economical distribution of material will be found when it is concentrated in a few large parts. To carry out this idea to the fullest extent, he thinks it might be worth while, in designing a large bridge, to adopt the "king post truss," using the same form of truss throughout the secondary systems. The most economical angle for the braces in such an iron bridge will be found when they make nearly a right angle with each other ($m = .96$). When their lengths are great in proportion to their diameters, they may be made of several braced columns like an iron pier.

If there should be plenty of head room under the bridge, it would be still more economical to turn this upside down, so as to put the long pieces in tension, using links, and the shorter ones in compression. (In this case $m =$ about .87.) This may be looked upon as a Bollman truss, with the rods at a more economical angle.



If there are rock abutments, to which the upper chord as well as the tension members can be fixed, a still greater saving in material can be made by making it of links so as to take tension, the only parts in compression being the vertical posts. The value of m can then be reduced to .5 or less. It is rarely, however, that the location would admit of such a bridge. It would, perhaps, be most economical to curve the tension member so as to make an inverted parabolic truss. We then have the following statement of the stresses, supposing that it rests at the ends on the abutments without being fixed to them, and that the diagonals take only tension:



Let $W =$ the uniform load on one panel.
" $W' =$ the variable load on one panel.
" $S =$ the span.
" $h =$ the depth of the truss at the middle of the span.
" $n =$ the number of panels, supposed to be even.

The maximum tension on $A\,B = \frac{W'\,S}{8\,h} \quad \frac{A\,B}{B\,C}$

The maximum tension on $C\,D = \frac{W'\,S}{8\,h} \quad \frac{D\,C}{B\,C}$

The maximum tension on $E\,F = \frac{W'\,S}{8\,h} \quad \frac{E\,F}{B\,C}$

&c. = &c.

The maximum compression on $B\,G = W + W'$

The maximum compression on $A\,C = \frac{W'\,S}{8\,h} \quad \frac{n-1}{n-3} \quad \frac{D\,E}{B\,C} + W$

The maximum compression on $D\,E = \frac{W'\,S}{8\,h} \quad \frac{n-2}{n-4} \quad \frac{F\,H}{B\,C} + W$

The maximum compression on $F\,H = \frac{W'\,S}{8\,h} \quad \frac{n-3}{n-5} \quad \frac{I\,K}{B\,C} + W$

The maximum compression on LB, BC, CE, EH, &c. $= (W + W')\, n \dfrac{S}{8\,h}$

The maximum tension on $LG = \dfrac{S}{8\,h}\,(W + W')\,n\,\dfrac{LG}{BL}$

The maximum tension on $GA = \dfrac{S}{8\,h}\,(W + W')\,n\,\dfrac{GA}{BL}$

&c. = &c.

If the chord could be fixed at the abutments, a saving of material could be effected by making the abutments take the tension transmitted from the curved lower member, so that the chord would only have to bear the stresses from variable loads. By fixing this chord likewise to the abutments, the only portions in the bridge to resist compression would be the posts, and we should probably have the most economical arrangement of iron in the bridge which it were possible to make. We then have for this case the stresses on the posts and diagonals, and curved portion the same as before, but in the chord a stress of tension of $\dfrac{S}{8\,h}$ $n\,W'$. The most economical height will be about from one-fourth to one-seventh of the span. The posts can be connected together for mutual stiffening. In such a bridge it would be very important to have the adjustment correct. This should be effected by loosening the braces and letting the weight of the bridge come entirely on the curved member, and then screwing the main braces, or those which incline downwards towards the middle, until they are somewhat tight, but not too much so, and finally screwing the counters until they come to a good bearing. If the truss were inverted so as to put the curved part in compression (the truss only resting on the abutments without being fixed to them), the stresses would be the same as before in amount in the arch and chord, but of opposite kinds. They would be of the same kind and amount on

the diagonals in each panel, but with the amount interchanged in each pair (or transpose the upper and lower letters in the last figure and use the same equations), but on the posts we should have both tension and compression, the maximum amount of the former being $W + W'$ on each one, and of the latter as follows:

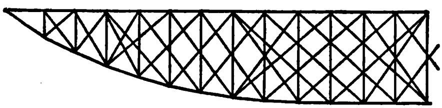
$$\text{Maximum compression on second post} = \frac{W' S}{8 h} \frac{1}{1 + 2} \frac{E D}{B C} - W$$

$$\text{Maximum compression on third post} = \frac{W' S}{8 h} \frac{2}{2 + 2} \frac{H F}{B C} - W$$

$$\text{Maximum compression on fourth post} = \frac{W' S}{8 h} \frac{3}{3 + 2} \frac{K I}{B C} - W$$

$$\&c. = \&c.$$

Returning to the arch in tension, it would be found that as the span became larger with a single system of intersection of the braces, the latter would become too much inclined towards the vertical for economy, and it would then be well to make the bridge with a double, treble, &c., intersection as in the figure.



This would, of course, complicate the stresses on some of the posts and braces above what has been given. In a bridge of several spans they could be connected together over the piers, so as to neutralize each other's weights. If the spans were long, the dead weight would probably so exceed the horizontal component produced on one side by a span being loaded when the adjacent one was unloaded, that the angle made on the pier by the resultant would differ so little from the vertical as to be well within the masonry without any special increase in the width of the piers.

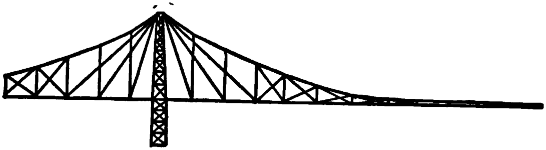
If the stress produced by the dead weight largely ex-

ceeded that produced by the variable load in any part, the factor of safety could be reduced on that part below the one-sixth recommended hereafter, since, of the two factors of which it is composed, as explained at the bottom of page 96, one of them supposes the whole load to be suddenly applied, while, in the case of a relatively heavy bridge, some parts will only receive a small proportionate increase of load suddenly. As to the method of finding the proper section in this case, the following formula for the unit stresses is given by Professor Merriman in Van Nostrand's Magazine, vol. 32, p. 98:

$$S = e\left(1 + \tfrac{3}{4}\frac{p}{P} + \tfrac{1}{4}\frac{p^2}{P^2}\right)$$

where S is the unit stress required to produce rupture when the applied stress ranges from p to P, and is applied a great number of times, and e is the elastic limit, which may be taken at 27,500 lbs. per square inch for wrought iron. Professor Merriman recommends a factor of safety of 4, but the writer thinks that 3 would be sufficient. In calculating the stresses in a large bridge care should be taken that the effect of temperature in changing the lengths of the several parts is considered.

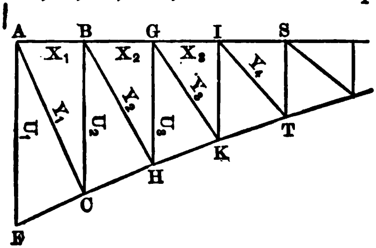
Where headroom under the bridge is important, a suspension bridge might be designed to act in a somewhat similar way to the parabolic arch described by making only the verticals take compression, and having the horizontal members connected together over the piers, and finally to

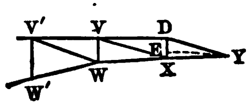


the abutments, so that a variable load shall be transmitted through them by tension. Of course the piers in such a bridge should be braced so as to resist the horizontal pull

at the top when one span is loaded and the adjacent one unloaded.

An economical form of arched truss is the following, supposed to be hinged at Y and F, and to have the points F, C, H, &c., in the arc of a parabola:





$A\,B = l.$ $\quad W =$ variable load on one panel.
$S =$ span. $\quad W' =$ uniform load on one panel.
$R =$ rise.

$$\text{Maximum stress on } X_1 = \pm \frac{1}{B\,C} \left\{ C + (2\,C - W\,l) + (3\,C - 2\,W\,l) + (4\,C - 3\,W\,l) + \text{\&c.} \right\}$$

until the quantity in the parenthesis (....) is about to become a minus quantity, in which

$$C = \left(\frac{\frac{1}{2}\,S - l}{S} + \frac{B\,C - D\,E}{2\,R} \right) W\,l.$$

$$\text{Maximum stress on } X_2 = \pm \frac{1}{G\,H} \left\{ C' + 2\,C' + (3\,C' - W\,l) + (4\,C' + 2\,W\,l) + \text{\&c.} \right\}$$

until the quantity in the parenthesis (....) is about to become a minus quantity, in which

$$C' = \left(\frac{\frac{1}{2}\,S - 2\,l}{S} + \frac{G\,H - D\,E}{2\,R} \right) W\,l.$$

$$\text{Maximum stress on } X_3 = \pm \frac{1}{I\,K} \left\{ C'' + 2\,C'' + 3\,C'' + (4\,C'' - W\,l) + (5\,C'' - 2\,W\,l) + \text{\&c.} \right\}$$

until the quantity in the parenthesis (....) is about to become a minus quantity, in which

$$C'' = \left(\frac{\frac{1}{2}S - 3\,l}{S} + \frac{I\,K - D\,E}{2\,R}\right) W\,l.$$

$$\&c. = \&c.$$

$$\text{Maximum stress on } Y_1 = \pm \frac{W}{L\,M}\Big\{\Big[\frac{l}{S}\left(\tfrac{1}{2}S - A\,L\right) - \frac{l}{2}\,\frac{D\,E}{R} + (A\,L - l)\Big] + \Big[2\,\frac{l}{S}\left(\tfrac{1}{2}S - A\,L\right) - 2\,\frac{l}{2}\,\frac{D\,E}{R} + (A\,L - 2\,l)\Big] + \Big[3\,\frac{l}{S}\left(\tfrac{1}{2}S - A\,L\right) - 3\,\frac{l}{2}\,\frac{D\,E}{R} + (A\,L - 3\,l)\Big] + \&c. - \&c. + \&c.\Big\}$$

until the sum of the terms in the parenthesis [....] is about to become a minus quantity, in which

$$A\,L = \frac{A\,F}{A\,F - B\,C}\,l, \text{ and } L\,M = \frac{B\,C}{A\,C}(A\,L).$$

$$\text{Maximum stress on } Y_2 = \pm \frac{W}{N\,O}\Big\{\Big[\frac{l}{S}\left(\tfrac{1}{2}S - A\,O\right) - \frac{l}{2}\,\frac{D\,E}{R}\Big] + \Big[2\,\frac{l}{S}\left(\tfrac{1}{2}S - A\,O\right) - 2\,\frac{l}{2}\,\frac{D\,E}{R} + (A\,O - 2\,l)\Big] + \Big[3\,\frac{l}{S}\left(\tfrac{1}{2}S - A\,O\right) - 3\,\frac{l}{2}\,\frac{D\,E}{R} + (A\,O - 3\,l)\Big] + \Big[\&c. - \&c. + \&c.\Big]\Big\}$$

until the sum of the terms in the parenthesis [....] is about to become a minus quantity, in which

$$A\,O = l + \frac{B\,C}{B\,C - G\,H}\,l, \text{ and } N\,O = \frac{G\,H}{B\,H}(A\,O - l).$$

$$\text{Maximum stress on } Y_3 = \pm \frac{W}{P\,Q}\Big\{\Big[\frac{l}{S}\left(\tfrac{1}{2}S - A\,Q\right) - \frac{l}{2}\,\frac{D\,E}{R}\Big] + \Big[2\,\frac{l}{S}\left(\tfrac{1}{2}S - A\,Q\right) - 2\,\frac{l}{2}\,\frac{D\,E}{R}\Big] + \Big[3\,\frac{l}{S}\left(\tfrac{1}{2}S - A\,Q\right) - 3\,\frac{l}{3}\,\frac{D\,E}{R} + (A\,Q - 3\,l)\Big] + \Big[\&c. - \&c. + \&c.\Big]\Big\}$$

until the sum of the terms in the parenthesis [....] is about to become a minus quantity, in which

$$A\,Q = 2\,l + \frac{G\,H}{G\,H - I\,K}\,l, \text{ and } P\,Q = \frac{I\,K}{G\,K}(A\,Q - 2\,l).$$

Maximum stress on $Y_4 = \pm \frac{W}{R\,U}\Big\{\Big[\frac{l}{S}(\frac{1}{2}\,S - A\,U) - \frac{l}{2}\;\frac{D\,E}{R}\Big] +$
$\Big[2\,\frac{l}{S}(\frac{1}{2}\,S - A\,U) - 2\,\frac{l}{2}\,\frac{D\,E}{R}\Big] + \Big[3\,\frac{l}{S}(\frac{1}{2}S - A\,U)$
$- 3\,\frac{l}{2}\;\frac{D\,E}{R}\Big] + \Big[4\,\frac{l}{S}(\frac{1}{2}\,S - A\,U) - 4\,\frac{l}{2}\;\frac{D\,E}{R} +$
$(A\,U - 4\,l)\Big] + \Big[\text{\&c.} - \text{\&c.} + \text{\&c.}\Big]\Big\}$

until the sum of the terms in the parenthesis [....] is about to become a minus quantity, in which

$$A\,U = 3\,l + \frac{I\,K}{I\,K - S\,T}\,l, \text{ and } R\,U = \frac{S\,T}{I\,T}(A\,U - 3\,l).$$

The calculation on these braces should be stopped when the value of the first term in the small parenthesis ($\frac{1}{2}\,S$—$A\;U$) in the last value, is about to become a minus quantity. The stresses in the remaining braces should be calculated from the middle as follows :

Maximum stress on $D\,Y = \pm \frac{W}{A'\,B'}\Big(\frac{1}{2}\,S\,\frac{\frac{1}{2}\,S - l}{l}\Big)\;\;\Big(1 + \frac{D\,E}{R}\Big)$

$$\text{in which } A'\,B' = \frac{R + \frac{\frac{1}{2}\,S}{l}(D\,X - D\,E)}{\frac{1}{S}\sqrt{S^2 + 4\,R^4}}$$

Maximum stress on $V\,X = \pm \frac{W}{A_2\,Z}\Big\{\frac{(\frac{1}{2}\,S - l)\,(\frac{1}{2}\,S - 2.l)}{2\,S\,l}\Big(D'\,Z +$
$\frac{S}{2\,R}\,D\,E\Big)\Big\}$, in which $D'\,Z' = l\Big(\frac{V\,W}{V\,W - D\,X} - 2\Big)$
and $A_2\,Z = \frac{D\,X}{V\,X}(2\,l + D'\,Z)$.

Maximum stress on V_1 $W = \pm \frac{W}{A_3 Z'} \Big\{ (D' Z' + l) - \frac{\frac{1}{2} S - l}{S}$
$D' Z' - \frac{\frac{1}{2} S - l}{2 R} (D E) + (D' Z' + 2 l) - \frac{\frac{1}{2} S - 2 l}{S}$
$D' Z' - \frac{\frac{1}{2} S - 2 l}{2 R} (D E) \Big\}$, in which $D' Z' = l$
$\left(\frac{V' W'}{V' W' - V W} - 3 \right)$ and $A_3 Z' = \frac{V W}{V' W}$
$(3 l + D' Z')$.

Maximum stress on next one $= \pm \frac{W}{A_4 Z''} \Big\{ (D' Z'' + l) - \frac{\frac{1}{2} S - l}{S}$
$D' Z'' - \frac{\frac{1}{2} S - l}{2 R} D E + (D' Z'' + 2 l) - \frac{\frac{1}{2} S - 2 l}{S}$
$D' Z'' - \frac{\frac{1}{2} S - 2 l}{2 R} D E + (D' Z'' + 3 l) - \frac{\frac{1}{2} S - 3 l}{S}$
$D' Z'' - \frac{\frac{1}{2} S - 3 l}{2 R} D E \Big\}$, in which $D' Z'' = l$
$\left(\frac{V'' W''}{V'' W'' - V' W'} - 4 \right)$ and $A_4 Z'' = \frac{V' W'}{V'' W}$
$(4 l + D' Z'')$.

Note that $D' Z'$ is an algebraic sum, and therefore may become a minus quantity. It must be treated with its proper sign in this case. About that time the amount obtained will become less than the amount obtained on the brace by approaching from the end of the bridge, when the calculation must be stopped.

Maximum tension on $U_1 = \frac{L M}{A L}$ (maximum on Y_1) $+ W + \frac{1}{2} W'$.

Maximum compression on $U_1 = \frac{L M}{A L}$ (maximum on Y_1) $- \frac{1}{2} W'$.

Maximum tension on $U_2 = \frac{N O}{B O}$ (maximum on Y_2) $+ W + \frac{1}{2} W'$.

Maximum compression on $U_2 = \frac{N O}{B O}$ (maximum on Y_2) $- \frac{1}{2} W'$.

&c., &c.

These calculations should be stopped at the same time that the corresponding calculations on the Y was stopped,

and the remainder calculated from the middle. We then have

Maximum compression on $D\,X = \frac{W}{D'\,Z^n + l}\Big\{\frac{(\frac{1}{2}S - l)(\frac{1}{2}S - 2l)}{2Sl}$
$\left(D'\,Z^n + \frac{2R}{S}D\,E\right)\Big\} + W + \frac{1}{2}W'.$

Maximum tension on $D\,X = \frac{W}{D'\,Z^n + l}\Big\{\frac{(\frac{1}{2}S - l)(\frac{1}{2}S - 2l)}{2Sl}$
$\left(D'\,Z^n + \frac{S}{2R}D\,E\right)\Big\} - \frac{1}{2}W$, in which $D'\,Z^n = l$
$\left(\frac{D\,X}{D\,X - D'\,Y} - 1\right)$.

Maximum compression on $V\,W = \frac{W}{D'\,Z + 2l}\Big\{(D'\,Z + l) - \frac{\frac{1}{2}S - l}{S}$
$D'\,Z - \frac{\frac{1}{2}S - l}{2R}(D\,E) + (D'\,Z + 2l) - \frac{\frac{1}{2}S - l}{S}$
$D'\,Z - \frac{\frac{1}{2}S - 2l}{2R}(D\,E)\Big\} + W + \frac{1}{2}W'.$

Maximum tension on $V\,W = \frac{W}{D'\,Z + 2l}\Big\{(D'\,Z + l) - \frac{\frac{1}{2}S - l}{S}$
$D'\,Z - \frac{\frac{1}{2}S - l}{2R}(D\,E) + (D'\,Z + 2l) - \frac{\frac{1}{2}S - l}{S}$
$D'\,Z - \frac{\frac{1}{2}S - 2l}{2R}(D\,E)\Big\} - \frac{1}{2}W.$

Maximum tension on $V'\,W' = \frac{W}{D'\,Z' + 3l}\Big\{(D'\,Z' + l) - \frac{\frac{1}{2}S - l}{S}$
$D'\,Z' - \frac{\frac{1}{2}S - l}{2R}(D\,E) + (D'\,Z' + 2l) + \frac{\frac{1}{2}S - 2l}{S}$
$D'\,Z' - \frac{\frac{1}{2}S - 2l}{2R}(D\,E) + (D'\,Z' + 3l) +$
$\frac{\frac{1}{2}S - 3l}{S}D'\,Z' + \frac{\frac{1}{2}S - 3l}{2R}(D\,E)\Big\} - W - \frac{1}{2}W.$

Maximum compression on $V'\,W' = \frac{W}{D'\,Z' + 3l}\Big\{(D'\,Z' + l) -$
$\frac{\frac{1}{2}S - l}{S}D'\,Z' - \frac{\frac{1}{2}S - l}{2R}(D\,E) + (D'\,Z' + 2l)$
$+ \frac{\frac{1}{2}S - 2l}{S}D'\,Z' - \frac{\frac{1}{2}S - 2l}{2R}(D\,E) + (D'\,Z' +$
$3l) + \frac{\frac{1}{2}S - 3l}{S}D'\,Z' - \frac{\frac{1}{2}S - 3l}{2R}(D\,E)\Big\} + \frac{1}{2}W'.$

Maximum tension on $V'' W'' = \frac{W}{D' Z'' + 4 l} \Big\{ (D' Z'' + l) - \frac{\frac{1}{2} S - l}{S} D' Z'' - \frac{\frac{1}{2} S - l}{2 R} (D E) + (D' Z'' + 2 l) - \frac{\frac{1}{2} S - 2 l}{S} D' Z'' - \frac{\frac{1}{2} S - 2 l}{2 R} (D E) + (D' Z'' + 3 l) - \frac{\frac{1}{2} S - 3 l}{S} D' Z'' - \frac{\frac{1}{2} S - 3 l}{2 R} (D E) + (D' Z'' + 4 l) - \frac{\frac{1}{2} S - 4 l}{S} D' Z'' - \frac{\frac{1}{2} S - 4 l}{2 R} (D E) \Big\} - W - \frac{1}{2} W'.$

Maximum compression on $V'' W'' = \frac{W}{D' Z'' + 4 l} \Big\{ (D' Z'' + l) - \frac{\frac{1}{2} S - l}{S} D' Z'' - \frac{\frac{1}{2} S - l}{2 R} (D E) + (D' Z'' + 2 l) - \frac{\frac{1}{2} S - 2 l}{S} D' Z'' - \frac{\frac{1}{2} S - 2 l}{2 R} (D E) + (D' Z'' + 3 l) - \frac{\frac{1}{2} S - 3 l}{S} D' Z'' - \frac{\frac{1}{2} S - 3 l}{2 R} (D E) + (D' Z'' + 4 l) - \frac{\frac{1}{2} S - 4 l}{S} D' Z'' - \frac{\frac{1}{2} S - 4 l}{2 R} (D E) \Big\} + \frac{1}{2} W'$

&c., &c.

Maximum compression on $F C = \dfrac{\frac{1}{8} \frac{S^2}{l} (W + W') \left(1 + \frac{D E}{R}\right)}{A C'}$

in which $A C' = \frac{A E}{F C} l.$

Maximnm compression on $C H =$

$$\frac{(\frac{1}{2} S - l) \frac{W l}{S} + \Big\{ (W + W') \frac{S^2}{8 R l} \Big\} D E + (W + W') \frac{(\frac{1}{2} S - l)^2}{2 l}}{B H'}$$

in which $B H' = \frac{B C \times A B}{C H}.$

Maximum compression on $H K =$

$$\frac{(\frac{1}{2} S - 2 l) \frac{(1 + 2 + 3) W l}{S} + \Big\{ (W + W') \frac{S^2}{8 R l} - \frac{(1 + 2 + 3) W l}{2 R} \Big\} D E + (W + W') \frac{(\frac{1}{2} S - 2 l)^2}{2 l} - W l,}{G K}$$

in which $G K = \frac{G H}{H K} l.$

Maximum compression on $K\,T =$

$$\frac{\left(\tfrac{1}{2}\,S - 3\,l\right)\dfrac{(1+2+3+4)\,W\,l}{S} + \left\{(W+W')\dfrac{S^2}{8\,R\,l} - \dfrac{(1+2+3+4)\,W\,l}{2\,R}\right\} D\,E + (W+W')\dfrac{(\frac{1}{2}S - 3\,l)^2}{2\,l} - W'\,l}{I\,T}$$

in which $I\,T = \dfrac{I\,K}{K\,T}\,l$, &c., &c.

This must be stopped a little beyond the quarter of the span, and the stresses of the remainder calculated from the middle. The greater value from either end to be taken in the panels near the quarter span.

Maximum compression on $X\,Y = \dfrac{(W+W')\left(1 + \dfrac{S^2}{8\,R\,l}\,D\,E\right)}{D\,X'}$,

in which $D\,X' = \dfrac{D\,X}{X\,Y}\,l.$

Maximum compression on $W\,X =$

$$\frac{\left(\tfrac{1}{2}\,S - l\right)\left(\tfrac{1}{2}\,S - 2\,l\right)\dfrac{W}{S} + \left\{(W+W')\dfrac{S}{8\,R\,l} - \dfrac{(\frac{1}{2}S - l)(\frac{1}{2}S - 2l)}{l}\,\dfrac{W}{4\,R}\right\} D\,E + (W+W')\left(l + \dfrac{2\,l}{2}\right)}{V\,W''}$$

in which $V\,W'' = \dfrac{V\,W}{W\,X}\,l.$

Maximum compression on $W\,W' =$

$$\frac{\tfrac{3}{2}\left(\tfrac{1}{2}\,S - 2\,l\right)\left(\tfrac{1}{2}\,S - 3\,l\right)\dfrac{W}{S} + \left\{(W+W')\dfrac{S^2}{8\,R\,l} - \dfrac{(\frac{1}{2}S - 2\,l)(\frac{1}{2}S - 3\,l)}{l}\,\dfrac{W}{R}\right\} D\,E + (W+W')\left(l + 2\,l + \dfrac{3\,l}{2}\right)}{V'\,W'''}$$

, in which $V'\,W''' = \dfrac{V'\,W'}{W\,W'}\,l.$

Maximum compression on next $=$

$$\frac{\tfrac{4}{2}\left(\tfrac{1}{2}\,S - 3\,l\right)\left(\tfrac{1}{2}\,S - 4\,l\right)\dfrac{W}{S} + \left\{(W+W')\dfrac{S^2}{8\,R\,l} - \dfrac{(\frac{1}{2}S - 3\,l)(\frac{1}{2}S - 4\,l)}{l}\,\dfrac{W}{4\,R}\right\} D\,E + (W+W')\left(l + 2l + 3l + \dfrac{4\,l}{2}\right)}{\text{lever arm}}$$

&c., &c.

If this form is inverted it will become a kind of suspension bridge, in which the amounts of stress will be the same as before, except that compression becomes tension and *vice-versa.*

On the Pennsylvania Railroad, the greatest variable load that can come upon one track is supposed to be $1\frac{1}{2}$ tons per lineal foot, and all bridges are calculated for this. That is, the length of a panel multiplied by $1\frac{1}{2}$ will give the value of $2\,W'$ in tons in the previous statement of the stresses in the Howe truss bridge. The uniform load, or weight of one panel of bridge, $2\,W$, for bridges over 50 feet span, may be calculated from the empirical formula:

$$2\,W = l\left[\frac{2+\frac{1}{18}S}{17+\dfrac{1}{\sqrt{S}}}\right]$$

in which l is the length of the panel in feet, S is the span in feet, and W is in tons.

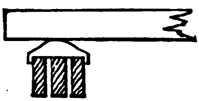
After the design of the bridge is complete, the actual weight should be calculated from the drawings, and a correction made, if necessary. Theory seems to say that the weight per foot lineal, is proportional to the span, and if the span is large, the second term in the denominator may be neglected in comparison with the first, and the formula would accord with theory. However, the above seems to suit actual examples better; probably because in smaller spans the material, especially at the joints, must be disposed according to laws which differ from that of varying as the square of the span. For instance, in an iron bridge, if the chord is thickened at the joint to distribute the pressure of the pin over a larger area, the increase of weight due to the thickening is proportional to the number of panels, and is not necessarily a function of the span.

So when pins are proportioned to resist the crushing stress into the chord, they will be larger than is necessary for shearing, which gives an excess of material in the axis

of the pin, which adds to the dead weight of the bridge, and is proportional to the number of panels, and likewise to the section of the chord, and independent to some extent of the span. The amount of material in the lateral and diagonal bracing will likewise increase with the distance apart of the trusses, even though the span remains the same.

The floor beams and track-stringers should be calculated for the greatest concentrated load which can come upon them, which is the weight on a pair of driving-wheels, and amounts to from 24,000 to 34,000 pounds. (Some of the later passenger engines even bring a load of 50,000 lbs. on a pair of drivers, a load of 100,000 lbs. in 15 feet, a load of 120,000 lbs. in 20 feet, and a uniform load of 3,700 lbs. per lineal foot for a length of 47 feet.)

When the floor beams rest on a chord, its resistance must be calculated for the transverse stress so produced, in addition to the stress calculated on the supposition that the load is concentrated at the panel points. The English generally only allow the load to come on the truss at the panel points. In America, however, the advantage gained by supporting the engine on the floor beams, in case of a derailment, has been considered sufficient to retain the method of resting them on the chord. It may sometimes be well, if the trusses are far apart, to rest them on a saddle, so as



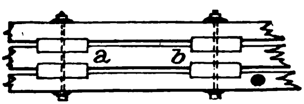
to insure their bearing equally on the chord, and not merely at the inside edge, when the floor beam deflects.

For calculating the transverse stress on the chord we use the formula for a beam fixed at the ends and uniformly loaded, that is,

$$\text{Breaking weight} = \frac{24\,S\,I}{d\,L}$$

in which S, d and l have the same values on page 61, and I is the moment of inertia or the value of r^2 in the table given hereafter multiplied by the cross-section. It is, however, uncertain how much this transverse stress would weaken the column, and the author thinks that the safe method would be to add a sufficient amount of material on the sides and top and bottom to resist the transverse load without any assistance from that in the column. Of course the proper section having been found in this way, it will be built up in the easiest manner without regard to what part acted for the axial and what for the transverse stress. It might be added that some actual bridges by prominent engineers seem to be wanting in a sufficient amount of material in the upper chord to resist the transverse stress.

Care should be taken that the resolved stress from a brace to a chord should pass through the center of gravity of the section of the chord, and similarly from a strut to a tie. In a Howe bridge, by varying the size of the angle block, the axes of the brace and rod can easily be made to pass through this centre of gravity, that is, intersect there. This same point of intersection of the three stresses in a Pratt bridge is the proper position for the pin. In reference to this pin, it may be remarked that in its manufacture the recommendations of Mr. Coleman Sellers in regard to axles should be observed; that is to say, turn it approximately true with a fine feed, and finish with a coarse feed revolving rapidly. This obviates the effect of the wear of the tool, which would otherwise make the surface conical in-



stead of cylindrical. In Howe truss bridges, the upper chord is formed of several sticks of timber, which are

made to act together by "keys" being notched into them. The distance apart of the keys, a b, should bear the same relation to the width of one of the sticks composing the chord as the length of one panel bears to the total width or depth (whichever is the least) of the chord. The depth of the notch is usually made $\frac{1}{8}$ the width of one of the sticks composing the chord. If the keys are made of cast iron, they are made cross-shaped, the distance across the shorter arms being one-third the width of one of the sticks, plus the distance apart of the sticks, the latter being the amount necessary to let the rods pass between; the longer arms are made twice as long as the shorter arms, and the thickness of each is equal to the space between the sticks. A bolt passes through the chord and the keys.

The strength of the upper chord in the panel, and likewise of the braces, should be calculated by Gordon's formula.

For a long column, fixed at the ends, we have

$$P = \frac{fS}{1 + a\,\frac{l^2}{h^2}}$$

where P is the breaking weight in pounds, l is the length and h is the least external diameter, both expressed in inches, S is the cross-section in square inches, $a = \frac{1}{250}$ for pine and $f = 5{,}000$ for pine. If, as is usually done, the braces are bolted to the counter-braces where they cross them, their breaking strength must be calculated as for a column of one-half the length, with one end fixed, and the other rounded, which may be taken as a mean of the strengths of two columns of this half length, one with both ends fixed, and the other with both ends rounded. The formula, however, for both ends rounded is:

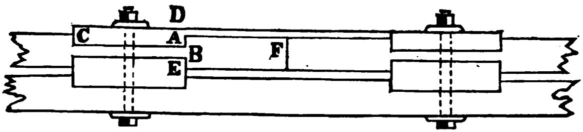
$$P = \frac{f S}{1 + 4a \frac{l^2}{h^2}}$$

It is only necessary to abut the several sticks of the upper chord against each other, without splicing them, making them break joints, however, so that there will not be more than one joint in a panel. The breaking weights calculated by the above formulas should be ten times the calculated maximum stresses in a wooden Howe truss bridge, except for the rods, which should be able to bear six times the maximum calculated stress. This is called the "factor of safety." The reason is believed to be as follows: Mr. Fairbairn found by experiment that when the load upon a piece of iron was equal to one-third of the breaking weight, as found by gradually applying heavier loads, and this was successively applied and taken off, the piece did not seem to have its ultimate resistance injuriously affected. If, however, the stress exceeded about one-third of this breaking weight, the piece would break after a sufficiently great number of applications of the load. It was also found by experiment, by an English Parliamentary Commission to test the value of iron for railroad construction, that a load suddenly applied to a bridge, as when an engine came rapidly upon it, produced double the amount of stress that would be produced by it if it were gradually applied. A multiplication of these two factors gives one-sixth of the breaking weight as found by a load gradually increased to breaking, as the greatest allowable stress to come upon a bridge which is subject to a rapid application of the load, applied many times. For wood, this constant is made one-tenth, since its strength varies very much in different specimens, and the experiments from which its strength has been found were made with selected pieces, of small size.

Instead of taking one-sixth of the ultimate strength for the safe load, it would no doubt be better to take one-half (or, safer, one-third) of the so-called "limit of elasticity," or the point at which successive applications of the same load produce an increasing set. Experiments on the strength of materials, however, have been heretofore directed rather to the ultimate strength, and this is what is recorded in the published tables. It may be observed, too, that the experiments from which the fact has been derived, of more than one-third of the breaking weight finally producing rupture, refer only to pieces strained in tension. It is probable that in compression a piece could sustain a repeated application of a greater proportion of the ultimate static load without rupture. Experiments are needed, however, to certainly establish this. In the Howe and Pratt bridges, each piece is only strained in one direction, either in tension or compression. In the Warren truss, however, the same piece may be strained sometimes in one way and sometimes in the other. The experiments of Wöhler tend to confirm the correctness of an American practice, that in this case such a piece should be proportioned to resist the maximum amount of compression; and should have, *in addition*, sufficient material to resist the maximum tension. The apparent advantage which the Warren has of requiring less pieces than the Pratt, and making one piece do double duty, is thus seen to be only apparent and not real, since the braces must contain an additional amount of material. This will perhaps explain why the Warren truss has never been so popular in the United States as it has been abroad.

In a Howe truss bridge the lower chord sticks have to be spliced, and the very common practice is to make the splice too weak.

The illustration shown at the top of the next page is, perhaps, the commonest form of such splice.



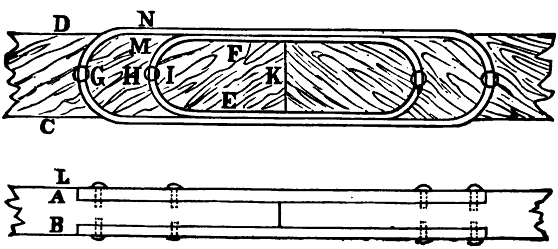
The notch $A\ B$ is generally too small, and bridges are often seen in which it is crushed. The proper relation between the parts of this splice is the following:

Let c = the crushing resistance of the wood = 5,000 lbs. per square inch for pine.
" t = the tensile resistance of the wood = 10,000 lbs. per square inch for pine.
" s = the shearing resistance of the wood = 600 lbs. per square inch for oak and 400 lbs. for pine.

Then

$$c \times 2\,A\,B = t \times B\,E = s \times 2\,A\,C = s \times 2\,B\,F = t \times A\,D.$$

For pine, we see that $A\ B$ will equal $B\ E$ and just two-thirds of the chord will have to be cut away. The splice-piece and keys are generally made of oak, or other hard wood. This is objectionable, since wood rots much more rapidly when it is in contact with a different species. A much better splice for the lower chord is that recommended by the late Mr. B. H. Latrobe, in the *Railroad Gazette*, vol. 10, p. 501. A flat iron link is set into the chord on each side. As a very small breadth is necessary to give the iron sufficient shearing resistance corresponding to the part $A\ C$, in the above wooden splice, sufficient space is



left on the chord to put two or more additional surfaces on the link to withstand the crushing force.

The following are the proper relations for determining the sizes :

$A\,B \times C\,D \times$ tensile resistance of the wood, should equal

$(C\,D + E\,F) \times A\,L \times 2 \times$ crushing resistance of the wood, and should also equal

$(G\,H \times C\,D + I\,K \times E\,F) \times 2 \times$ shearing resistance of the wood, and should also equal

$A\,L \times M\,N \times 8 \times$ tensile resistance of the iron bands.

Other forms of splices may be seen in the *Railroad Gazette*, vol. 10, p. 573, in an article by Mr. Geo. L. Vose.

When cast-iron keys are used, neither of the splices figured is of much value, as the superior rigidity of the key makes it do most of the work before the resistance of the splice can come into play. We must then rely on the resistance of those sticks in any one panel of the lower chord, which have no splices.

The amount that the angle blocks are notched into the chord is a point to which not sufficient attention has been given in many existing bridges. They should have sufficient bearing surface to resist the crushing stress transmitted from the brace to the chord. It fortunately happens that this is greatest at the ends, where there is an excess of material in the chords, as they are made of the same section throughout, generally, in wooden bridges. The necessary amount of notching, of course, diminishes to the middle.

The nuts at the ends of the rods in a Howe truss bridge rest on wrought-iron pieces, which act the part of washers to distribute the load over a considerable surface of the wood, to prevent crushing of the fibre. Care should be taken to have these sufficiently large to bring the crushing resistance of the wood well within its ultimate resistance. Cast-iron "tubes" are generally used to transmit

the stress from the rod to the angle block. They are ordered about a half-inch shorter than the depth of the chord, to allow for shrinkage in the latter. Particular attention, too, should be paid to having them strong enough to transmit the compression from the nut on the end of the rod to the angle block. The rods are generally ordered of such a length that they will project beyond the nut a distance equal to half the diameter of the rod. The joints of the chords, especially those of the lower chord, should be painted with red lead.

Provision should be made in the angle blocks, when of cast iron, for the inserting of dowel pins to hold the braces and counters in place until the rods are screwed up. They may be made of short lengths of iron rods three-quarters of an inch in diameter and six inches long. If wooden angle blocks are used, which, however, is objectionable, nailing the braces to them is sufficient.

In Howe truss bridges the parts *A F*, *A B* and *A G* in the figure on page 60 being without direct stress, can be left out. This is seldom done, however, since the counter-brace serves to stiffen the main brace by shortening its length as a column. If the bridge is a deck bridge, these pieces serve to carry the track over the space *A B* to the abutment. *A F* is then made of wooden struts strong enough to resist the half load on a panel, and the chords are held against the struts by rods.

Lateral and diagonal bracing is recommended to be of sufficient strength to resist a pressure of wind of 30 pounds per square foot of truss and train, using a factor of safety of one-sixth. In the tropics, where hurricanes occur, it should be calculated for a pressure of at least 50 pounds per square foot. Such has been the prevailing practice in the United States. Mr. C. Shaler Smith has, however, noted a pressure of 84 pounds at the St. Charles bridge, and a velocity of 138 miles per hour has been likewise noted. The latter

reduced to pressure by Smeaton's rule $p = \frac{v^2}{203\frac{1}{4}}$ gives a pressure of 93 pounds.

It is therefore recommended that in exposed situations it should be calculated for 100 pounds per square foot of bridge and train, and regarding, in an uncovered bridge, that one side does not shield the other, and likewise treating the portion on the train as a moving load. Considering, however, the rarity of such hurricanes, it may, perhaps, be allowed to use a factor of safety of 4. It should be stated however, that some modern experiments seem to indicate that Smeaton's rule gives pressures about double what they should be. See Mr. Wellington's experiments, Trans. Am. Soc. of Civil Engineers, vol. 8, p. 45.

The lateral bracing is generally made of the Howe or Pratt truss form, and since the stress may come upon it in either direction, the braces and counters are of the same size, acting alternately as one or the other, according to the direction of the wind. It should be observed that the stress produced by the wind on the lower chord, on the windward side, is compressive, which is antagonized by the tensile stress due to the load. The latter will generally be much the greater except in the end panels, which often require to be stiffened to resist the former.

Attention should be paid to the method of transferring the stresses from the angle blocks of the lateral bracing to the chords, and from the diagonal bracing to the rods. It may often be necessary to use thicker sticks in the lower chord to resist this stress, if it is requisite to notch the angle blocks in, to obtain sufficient bearing surface. For small spans, however, the bearing of the angle blocks on the rods, which bear on the chords, may be sufficient to resist this stress without notching.

For through bridges there should be 13 feet in the clear

between trusses. In deck bridges the trusses are not usually placed nearer together than $9\frac{1}{2}$ feet between centres. In long spans they should be placed farther apart to increase their stability against wind pressure. In the Ohio River Bridge at Cincinnati, of 515 feet span, the trusses are 20 feet apart, centre to centre.

In erecting Howe truss bridges, after the pieces of a truss are asembled on the false works in their proper positions, the rods are finally screwed up from both ends toward the middle. In Pratt truss bridges, the main braces are first screwed tight, those that are symmetrically placed on each side of the centre being screwed up together, and advancing from the middle toward the ends. The counters are afterwards screwed up, their adjustment being determined by the note which is given out when they are struck with a hammer. When in a Pratt bridge the main braces are not adjustable, but are formed of eyebars, the lower chord pieces must be very accurately placed at their proper cambered height on the false works in erection. The posts and braces are erected first at the middle, and then successively to the ends. It is better that the chord pieces should be a little too high than the least too low; for if the latter should prove to be the case, the whole bridge would have to be raised bodily in order to get the end braces in place; while if a little too high, the blocks can be chipped or knocked out, when the end is reached, until the braces can be sprung to their places.

On the Pennsylvania Railroad the floor system designed by Mr. Joseph M. Wilson consists of white oak floor beams 7 inches wide by 12 inches deep, placed 15 inches apart, centre to centre, resting on the chords and notched half an inch on them. The rails are spiked directly to these floor beams, and, outside of them, longitudinal guard timbers of white oak 6 inches square are placed, notched one inch on the floor beams, bolted to them at every fourth beam with

one inch bolts with flat heads, and spiked at the intermediate ones. The chord must be calculated to resist the transverse stress thrown upon it by these floor beams, in addition to the stress obtained from the strain sheet. On the inside of each rail, a guard rail is likewise fastened to the floor beams, close to the traffic rail. At the end of the bridge these guard rails should approach each other, so as to meet at a distance of about 30 feet. On the New York, Pennsylvania and Ohio Railroad the bridge floor consists of floor-beams 9 inches wide by 6 inches deep, spaced 12 inches apart, centre to centre, resting on iron stringers under each rail, and also at the outside of the bridge, with a guard timber 6 × 5 notched 1 inch, bolted to each fifth beam at a distance of 8 inches from the rail for through bridges and over the outside trusses for deck bridges. (See *Engineering News*, Feb. 12, 1887, for another method.)

For bridges of twenty-feet span, wrought-iron built I-beams of 12 inches depth are used, two to each rail. On these cross-ties, 6 × 8, 7½ feet long, are notched one inch, spaced two feet apart; a stringer, 6 × 12, notched half an inch, and four feet longer than the iron beams, rests upon the cross-ties, and supports the rails.

It is found that when beams are notched as described above, they are very liable to split from the notch along the grain of the wood. It would no doubt be better to use some other method of keeping them in place, such as fastening blocks to them.

Back-wall plates, 6 × 12: The coping should be strong enough to resist the pressure without wall-plates, using a cast-iron shoe 1½ inches thick, which should be large enough to distribute the load so that not more than 25,000 pounds per square foot come on the coping.

Bridges are always built with the upper surface slightly convex, so that when they are loaded the track shall not be below a horizontal line joining the extremities of the

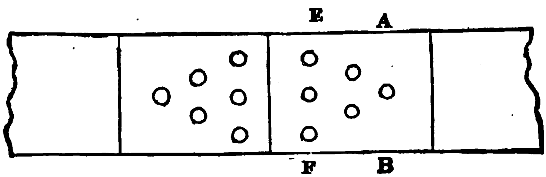
span. This rise is called the camber, and it is put into a bridge by making the upper chord longer than the lower one, and likewise making the main bracing (if not adjustable) to suit this additional length.

The additional length is $= \frac{(a + a') S}{E}$ in which a and a' are the maximum safe compressive and tensile stresses on the material, per square inch, S is the span, and E is the modulus of elasticity. For wood $a + a'$ may be taken at 2,000 and E at 1,600,000; for iron $a + a'$ is about 19,000 and E is 22,000,000. The additional length of chord thus found must be divided up among all the panels. By making the substitutions it is seen that the upper chord of a wooden bridge must be in the proportion of 1½ inches longer than the lower one for each 100 feet of span; and of an iron bridge one inch longer. If the height of truss is one-eighth of the span, the additional length of the chord will equal the camber. There is some reason for thinking that the most economical height for a truss (regarding only its resistance to the vertical load, and not to the wind pressure), is from one-sixth to one-tenth the span; and the most economical angle for the inclined braces is about 45° with the vertical. To obtain both of these advantages with a small panel length, it is necessary in long spans to have two or more systems of braces and the bridge is then called double or treble, etc., intersection. It would perhaps, however, be productive of the greatest economy to use only one system of bracing with the above mentioned height and angle, and use smaller trusses to span the distance between panel points. The material would then be concentrated in a few large parts. It should be observed, however, that many engineers, perhaps most of them, would disagree with the author on this point,

although he thinks the prevailing tendency of modern bridge engineering is in that direction.

RIVETTING.

In bridge construction rivets have, according to their positions, two functions to perform; first, to transmit a tensile stress from one piece to another, and second to cause two or more pieces in compression to act together, so as to give a united resistance. The first function is the only one that is required in boiler construction, and the rules that are generally given for rivetting are derived from experiments made by boiler makers. When two pieces in tension are fastened with rivets in bridge construction, they are always "double rivetted," or have a cover plate rivetted on each side of the plates that are connected. The following is a sketch of this mode of construction.



This joint may give way in either of the following manners: 1st. By tearing across the plate at *A B*. 2d. By tearing across the two cover plates at *E F*. 3d. By shearing the rivets. 4th. By the rivets crushing into the plate.

Let t = the thickness of the plate in inches.
" t' = " " " each cover plate in inches.
" w = " width of the plates in inches.
" d = " diameter of the rivets in inches.
" n = " number of rivets in the row nearest the joint, as row *E F*.
" T = " tenacity of the iron = 48,000 lbs. per square inch. (It should elongate fifteen per cent. before breaking).

Let S = the shearing resistance of the iron = 40,000 lbs. per square inch.

" C = " crushing resistance of the iron = 20,000 lbs. per square inch.

This last value is derived from experiments on pin connections. Experiments on rivetted connections seem to make it as high as 80,000. It is probable however, that the friction between the rivet head and the plate increased the strength in the experiments, so that the rivet did not bear against the plate with its length only. In bridge construction the vibration of the bridge would probably render unreliable this friction, so that it is better to take a smaller value and use more rivets.

The equations for equal strength are

$$T(w-d)\,t = 2\,T(w-nd)\,t' = \frac{\pi d^2}{4}\,2\,S\,\frac{n}{2}\,(n+1) = t\,d\,\frac{n}{2}$$

$$(n+1)\,C = 2\,d\,\frac{n}{2}\,(n+1)\,t'\,C.$$

Rivets with sufficient bearing surface always have enough shearing area. We may then take the above equations

$$T(w-d)\,t = 2\,T(w-n\,d)\,t' = t\,d\,\frac{n}{2}\,(n+1)\,C$$

from which we deduce :

$$n^2 + n = 4.8\left(\frac{w}{d} - 1\right)$$

and

$$t' = \frac{1}{2}\,\frac{w-d}{w-n\,d}\,t$$

n, d and t are the values usually given. Ordinary values of d range from ⅜ to 1¼ (or 1½ inches in shipbuilding) inches, and such a value between these limits is chosen as will make $d = 1.1\,\sqrt{t}$ if possible. (M. Antoine's rule.)

A rivet may also shear the metal on each side ahead of it if there is not sufficient distance between it and the end of the plate, or it may cause the metal to tear at the out-

side opposite its centre, and at its side. We may regard the latter as the case of a beam uniformly loaded and fixed at the ends. If the resistances to these are put equal to the resistance to crushing into the plate, we have

$$t\,d\,C = (l + \tfrac{1}{2}\,d)\,2\,S = \tfrac{1}{6}\,K\,t\,\frac{l^2}{d}$$

in which l is the distance from the outside of the first row of rivets, $E\,F$, to the edge of the plate, and K is the S of page 61 $= 40{,}000$ for wrought iron. From these equations we have

$$l = \frac{d}{4}\,(t - 2) \text{ and } l = 1.73\,d.$$

For ordinary cases the first will give a negative value, which shows that the joint will not fail by shearing the plate, and the second one is then to be generally used.

The rows are usually placed at such a distance apart that the rivets form equilateral triangles with each other.

When several plates are rivetted together, they should not be in thickness more than five times the diameter of the rivet.

For pin connections $n = 1$, and the equation on the last page gives

$$d = \tfrac{3}{4}\,w.$$

The results of experiments in England, according to Mr. Berkeley, show that the following are the relations of the dimensions for the strongest form.

Bar width $= 1$; diameter of pin $= \frac{3}{4}$; sum of sides of eye $= 1\frac{1}{4}$; end of eye $= 1$; radius of shoulder $= 1\frac{1}{2}$ (to 2).

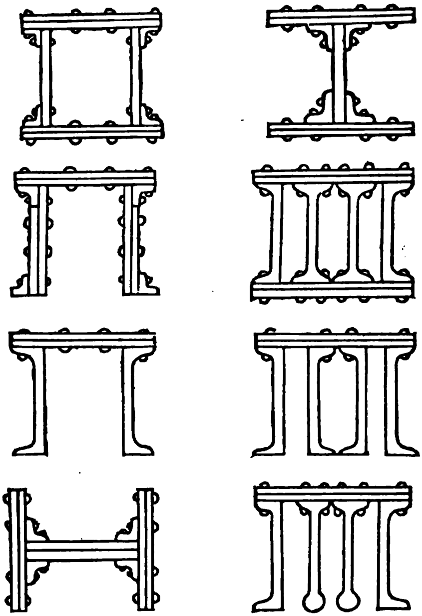
According to experiments by Mr. C. Shaler Smith (Trans. Amer. Soc. of Civil Engineers, Sept. 1877) it seems that when several bars of varying widths come upon the same pin, the latter should be of a diameter not less than two-thirds

of the width of the largest bar, the section across the side of the eye should be 1.33 for hammered, and 1.5 the width of the bar for "weldless" eyes; and the maximum thickness of these bars should not be more than one-fourth of their width. The metal section across the eyes in the smaller bars, and *their* thickness, should bear the following relation to the width of the bar:

Diameter of pin divided by the width of the bar.	Hammered Eyes.		Weldless Eyes.	
	Section across side of eye.	Max. thickness of bar.	Section across side of eye.	Max. thickness of bar.
$\frac{2}{3}$	$1\frac{1}{3}$	.21	1.5	.21
$\frac{3}{4}$	$1\frac{1}{3}$	.25	1.5	.25
1	$1\frac{1}{2}$	.38	1.5	.38
$1\frac{1}{4}$	$1\frac{1}{2}$	.54	1.6	.54
$1\frac{1}{3}$	....	....	1.7	.59
$1\frac{1}{2}$	$1\frac{2}{3}$	.70	1.85	.70
$1\frac{3}{4}$	$1\frac{2}{3}$	.88	2.	.88
2	$1\frac{3}{4}$	1.08	2.25	1.08

If the bar is "upset" at the ends, for t in the second member of the equation on the last page substitute $t'=$ the thickness of the upset part. In iron bridges the American practice is to make the portions which bear tension of eye bars. The parts in compression are universally made, when wrought iron is used, of members "built up" of several pieces of various sections, rivetted together. When compression members are made up in this way, the centre of gravity of the united section should coincide with the centre of figure; this can generally be accomplished by a manipulation of the parts in the design. In building up sections in this way, the rivets are usually spaced about 5 diameters apart.

The following are some sections of upper chords:



For calculating the strength, Rankine's formula, in the following form, for columns fixed at the ends, should be employed:

$$\frac{P}{S} = \frac{f}{1 + a\,\frac{l^2}{r^2}}$$

where P is the breaking load in pounds, S is the cross-section in square inches, l is the length in inches, r is the "least radius of gyration" in inches, f is the crushing resistance of the material, taken by Rankine at 36,000 pounds for wrought iron, but which for American iron made by the best manufacturers may be taken at 40,000

pounds, and $a = \frac{1}{25000}$. An approximate formula when $\frac{l}{r}$ is not more than 63 is

$$\frac{P}{S} = 86\left(462 - \frac{l}{r}\right)$$

For cast iron f may be taken at 80,000 and a at $\frac{1}{800}$, if we substitute the least diameter, h for r. (If the ends are rounded, or consist of eyes resting on pins, 4 a should be substituted for a.) The column is then treated as a whole, and in order that it may so act, the rivets connecting the parts should be so spaced that their distance apart bears the same or a less proportion to the thickness of either of the parts connected as the least diameter of the chord (or column) bears to its length between panels. A factor of safety of one-sixth should be employed. The value of a in the equation given by Prof. Rankine is usually taken at $\frac{1}{36000}$, a value derived from experiments on rectangular columns, but the value of $\frac{1}{25000}$ seems to agree with the American experiments when $f = 40,000$. A sufficient number of experiments on wrought iron in various shapes has not yet been made, and such experiments are perhaps the most important want that exists in the engineering profession, since "built" columns have come into universal use. The values of r^2 can be readily calculated from the formulas in the table.

Mr. Burr's formula for the strength of columns is

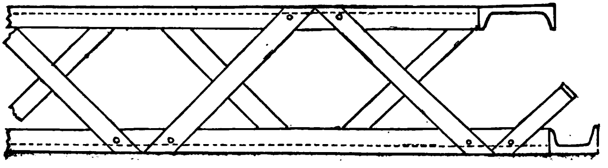
$$\frac{P}{S} = \frac{c}{\left(\frac{l}{r}\right)^b}$$

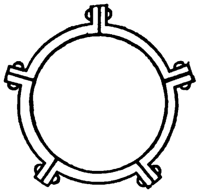
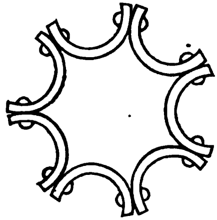
For "Phœnix" columns $c = 65,354$, and $b = .138$; for hollow cylindrical wrought iron columns $c = 116,390$ and $b = .317$.

The value of r in the formula, for any irregular section, if it is symmetrical with reference to two axes at right angles to each other, may be found by experiment thus. Draw the section on a large scale on cardboard; cut it out; suspend it by a pin thrust through it, near the edge, on a principal axis, and set it oscillating. Count the number of oscillations in a minute, and substitute in the formula:

$$r^2 = \tfrac{1}{2}\left(\frac{140,796}{N^2}e - e^2\right)$$

in which N is the number of oscillations, and e is the distance from the point of suspension to the centre of gravity of the section, in inches. The cardboard should be heavy, otherwise it may be difficult to get it to continue to oscillate for so long a time as a minute. The oscillations in half a minute multiplied by two would of course be the same, but it is difficult to count them exactly, and any error is doubled in the result. For columns acting as braces, a similar process may be employed. However, their sections are not usually continuous; for example, the "Phœnix" column, No. 4 in the previous table, sometimes has spaces left between the sections which compose it, occupied by "thimbles" or iron tubes around the rivets. In cutting out the cardboard figure in such a case, the places occupied by the rivets should be represented merely by a narrow piece, sufficient only to hold the sections so that they shall oscillate together. The following are some other forms of bracing for sustaining compression.



In pin connected bridges, it will often be necessary to rivet extra pieces to the chord around the pin holes, to distribute the stress over a large surface, so that the chord shall not fail by the pin crushing into it. Where a brace sustaining compression is composed of two channel irons, connected by pieces rivetted diagonally to the flanges, the least radius of gyration may be found by the same process with the card-board, substituting the narrow strips however, to represent the diagonal pieces, in the cross section. The diagonal pieces may then be proportioned so that the column may be equally strong in both directions, to a transverse strain. When the channels are turned back to back the least radius of gyration can be calculated from Case 6 in the table by making $l_1 = 0$. The lacing should be then calculated of a sufficient strength as above.

When a compression member is formed of two channel bars, or other pieces that are intended to act together, a piece of plate iron should be rivetted on both sides of the pieces at the ends, to ensure their acting together. When the flanges of channel bars have to be cut away at the ends to make a close fitting joint with a chord or post, special care should be taken that the strut is not too much weakened thereby.

Higher prices are usually charged for plate iron with an area of more than 30 square feet or a weight of more than 6 cwt. In bridge work it should never be less than a quarter of an inch thick. Usually it is not less

than three-eighths of an inch. It can be obtained up to $1\frac{1}{2}$ inches thick. I beams are made of heights from 4 to 20 inches, with areas from 3 to 27 square inches, and in lengths of 40 feet. Channel irons are from 6 to 15 inches high, with areas from 3 to 21 square inches, and lengths of 30 feet for 9 inches or less, and 25 feet for 12 inches, and 20 feet for 15 inches high. Deck beams are of heights from 5 to 12 inches, with areas from $3\frac{1}{2}$ to 14 square inches, and 30 feet long. Angle irons are from $1\frac{1}{2} \times 1\frac{1}{2} \times \frac{1}{4}$ to $6 \times 6 \times \frac{7}{8}$, with an area from $\frac{7}{10}$ to 9 square inches, and 25 feet long. In iron of these various sections, a higher price is usually charged if the section is more than 8 square inches or the weight more than 4 cwt. Rods of a less diameter than 1 inch should not be used, even though the stress sheet should indicate that smaller ones would be sufficient.

To illustrate the best practice of iron bridge building, drawings of the details are given of the bridge over the Ohio at Cincinnati, of 515 feet span, designed by Mr. Jacob H. Linville, of Philadelphia.

Particular attention is called to the following points: The pins in the upper chord are above its centre-line, so as to be nearer the centre of inertia of the cross-section of the chord in the middle of the bridge where the stress upon it is greatest, the additional section being formed by rivetting plates on top. The connection of the end post with the chord is formed by a pin only, round which the pieces will revolve on any deflection of the bridge. The load is suspended from the truss only at the panel points, and separate chords are used for the lower lateral bracing. The columns are stiffened in the middle of their lengths by longitudinal channel irons, which do not bear any of the principal stresses of the truss, but merely shorten the lengths of the columns. The two columns at each panel are made to act together by a piece of plate iron connecting them at their ends. The upper chord is

thickened at the joints with pieces of plate iron in order to distribute the stress on the pin over a larger surface. The only adverse criticism to be made is that the upper lateral and diagonal bracing do not fulfill the rule of their respective components of stress acting through the centres of inertia of the chord and post; therefore producing a bending stress on the pin and eccentric stresses on the compression parts. However, their stresses are small compared to those in the truss due to its load, and it is difficult to design a detail to accomplish perfectly the centering of all the stresses.

When the additional thickness required towards the middle of the span in the upper chord is obtained as in this case, by adding plates on top, if all the pin holes are drilled at the same distance from the bottom of the chord, some of them will not be at the centre of inertia of the cross-section, and will produce an eccentric stress in the chord. To place each pin at its proper place would increase the cost of manufacture by sacrificing uniformity. This difficulty is therefore got over in one of two ways: either the pin is put at the centre of inertia of that part of the chord where the stress is greatest and the bending stress at the other points calculated, and a sufficient additional amount of material added, above what is called for by the stress sheet, to neutralize the injurious effect; or else the better way is adopted of only having one plate on top, and getting the additional section required in the middle panels by plates on the sides. These plates are put so as to have the same cross-section both above and below the horizontal plane through the neutral axis of the chord, and so will neither raise nor lower the centre of inertia of any section of the chord. One end of an iron bridge, if of more than 50 feet span, should always rest on rollers, so that it can adapt its length to changes of temperature. To obtain the number of rollers of a given diameter to use,

we have Professor Burr's formula for the total weight on each one equal to

$$R\, l \sqrt{2\, w^2 \frac{E + E'}{E\, E'}}$$

in which R is the radius and l is the length of each, E and E' are the moduli of elasticity of the roller and plate which rests on it, and w is the greatest intensity of pressure on the roller. If we make $w = 12,000$ and $E = E' =$ 22,000,000 for iron, we have the safe weight on each roller equal to 560 $R\, l$; and dividing this into the total weight of one-half of truss with its load, gives the number of rollers. They are not usually made of less than 2 inches in diameter. E for soft steel is from 28,000,000 to 30,000,000.

The following is the specification of Messrs. Wilson, Brothers & Co., of Philadelphia, for wrought iron bridges :

"WROUGHT IRON.

"All wrought iron must be tough, fibrous, uniform in quality throughout, free from flaws, blisters and injurious cracks, and must have a workmanlike finish. It must be capable of sustaining an ultimate stress of forty-six thousand (46,000) pounds per square inch on a full section of test piece, with an elastic limit of twenty-three thousand (23,000) pounds per square inch.

"All iron to be used in tension or subjected to transverse stress (except web plates) must have a minimum stretch of fifteen (15) per cent., under ultimate stress, measured on a length of eight (8) inches.

"All iron to be used in compression and for web plates of width not exceeding twenty-four (24) inches, must have a minimum stretch of ten (10) per cent. under ultimate stress, measured on a length of eight (8) inches.

"All iron for web-plates exceeding twenty-four (24) inches

in width must have a minimum stretch of five (5) per cent. measured in length of eight (8) inches.

"All iron to be used in the tensile members of open trusses, laterals, pins, bolts, etc., must be double rolled after and directly from the muck bar (no scrap will be allowed) and must be capable of sustaining an ultimate stress of fifty thousand (50,000) pounds per square inch on a full section of test piece, with an elastic limit of twenty-five thousand (25,000) pounds per square inch and a minimum stretch of twenty per cent. in length of eight inches under ultimate stress.

"When tested to the breaking, if so required by the engineer, the links and rods must part through the body and not through the head or pin hole. Such tests must be at the expense of the contractor when the requirements of these specifications are not complied with.

"All tension wrought iron, if cut into testing strips one and a half ($1\frac{1}{2}$) inches in width, must be capable of resisting without signs of fracture, bending cold by blows of a hammer until the ends of the strip form a right angle with each other, the inner radius of the curve of bending being not more than twice the thickness of the piece tested. The hammering must be only on the extremities of the specimens, and never where the flexion is taking place. The bending must stop when the first crack appears.

"All tension tests are to be made on a standard test piece, of one and a-half ($1\frac{1}{2}$) inches in width, and from one-quarter ($\frac{1}{4}$) to three-quarter ($\frac{3}{4}$) inches in thickness, planed down on both edges equally, so as to reduce the width to one (1) inch for length of eight (8) inches. Whenever practicable, the two flat sides of the piece to be left as they come from the rolls. In all other cases, *both* sides of the test pieces are to be planed off. In making tests the stresses are to be applied regularly, at the rate of at least one (1) ton per square inch in fifteen seconds of time.

"All plates, angles, etc., which are to be bent in the manufacture must, in addition to the above requirements, be capable of bending sharply to a right angle, at a working heat, without showing any signs of fracture.

"All rivet iron must be tough and soft, and pieces of the full diameter of the rivet must be capable of bending until the sides are in close contact, without showing fracture on the convex side of the curve. Pins of 4½ inches in diameter or less may be rolled iron, but those of a greater diameter must be forged.

"*Workmanship.*—All workmanship must be first class; all abutting surfaces, except flanges of plate girders, must be planed or turned, so as to insure even bearings, taking light cuts so as not to injure the end fibres of the piece, and must be protected by white lead and tallow. Abutting surfaces must be brought into close and forcible contact when fitted with splice plates, and the rivet holes reamed in position before leaving the works, the plates being marked so as to go in the same position in erecting. Generally the use of bolts instead of rivets will not be permitted, unless they are turned conical and the holes reamed to fit them. Rollers must be turned and roller-beds planed.

"Rivet holes must be carefully spaced and punched, and must in all cases be reamed to fit where they do not come truly and accurately opposite, without the aid of drift pins. Rivets must completely fill the holes and have full heads, and be countersunk when so required and machine-driven wherever possible. Compression members must be straight and free from kinks or buckles in the finished piece.

"All pin holes in pieces which are not adjustable for length must be accurately bored at right angles to the axis unless otherwise shown on the drawings, and no variation of more than one sixty-fourth of an inch will be allowed in the length between centres of pin holes. Eye-bars

must be perfectly straight before boring ; the holes must be in the centre of the head and on the centre line of the bar. Whenever links are to be packed more than one-eighth of an inch to the foot of their length out of parallel with the axis of the structure, they must be bent with a gentle curve until the head stands at right angles to the pin in their intended position before being bored, suitable blocking pieces being used to keep them in proper position during the operation of boring. All pieces must be at equal temperature when bored, and those belonging to the same panel, when placed in a pile, must allow the pin at each end to pass through at the same time without forcing. Pins must be carefully turned, perfectly finished and straight, and when driven in must have pilot nuts to preserve threads. No variation of more than one thirty-second of an inch will be allowed between diameter of pin and pin hole. In the case of bolts, a variation of one sixteenth of an inch will be allowed between diameter of bolt and hole. Thickening washers may be used whenever required to make the joints snug and tight.

"All iron must receive one (1) coat of raw linseed oil as soon as received at the works, and a coat of approved red oxide of iron before leaving works. All inaccessible surfaces are to be painted with one (1) heavy coat red oxide of iron in pure linseed oil. All iron to be scraped clean from scale before painting.

"GENERAL CONDITIONS.

"The whole of the construction to be first-class work, and in strict accordance with the drawings and these specifications. In the case of sub-contractors, the specifications are fully binding on them in every respect, and free access and information is to be given by them for thorough inspection of material and workmanship, and all required

test pieces, etc., properly shaped, are to be provided as may be requested without charge. All shipments of material not properly inspected are at the risk of the contractor.

"In all cases figures are to be taken in preference to any measurements by scale.

"No alterations are to be made unless authorized by the engineers."

In a discussion of these specifications, Mr. Wm. Sellers thought that the clause "must be double rolled after and directly from the muck bar (no scrap will be allowed)" should be left out, it being the manufacturers' province alone to say how the material to meet prescribed tests shall be made. He thinks, too, that test pieces, instead of being a uniform length, should be of such a length that its ratio to the shortest diameter shall be constant; thus making the tests of different sizes comparable.

Bessemer and open-hearth steels are now coming into extensive use for bridge work. The utmost care is required, in making them, to avoid a greater heat than a dull red. Eye-bars ought to be finished at one heat, and afterwards well annealed in a furnace that will take in the whole bar at once.

Punching steel injures it much more than punching iron, and therefore when it is used for compression members the holes should be drilled. The plant of existing bridge companies being adapted to punching iron, an objection is made by them to drilling the pieces, and some engineers at present only use steel for tension members. Some manufacturers punch the holes small and ream them to the proper size. This removes the ring of metal injured by punching, and is almost as good as drilling the holes.

There is also a difference in regard to the amount of stress put upon the steel. At first the tendency was to use a somewhat hard steel with a high ultimate strength, but

now the tendency is to use a quality having an ultimate strength of 60,000 to 68,000 pounds per square inch, an elastic limit of 34,000 to 42,000, a stretch of 20 per cent. in 8 inches, and a reduction of area on breaking of 30 per cent. Full sized steel eye bars for the Hawksbury bridge had an ultimate strength of 66,000 pounds per square inch, an elastic limit of 36,000, and an elongation of 12 inches, or 37 per cent., with a reduction of area at fracture of 51 per cent. Small test pieces of $\frac{3}{4}$-inch round steel, on account of the greater amount of work expended in rolling them, will usually bear about 4 per cent. more than larger pieces, an elastic limit of 10 to 20 per cent. more, the elongation will be about the same, and the reduction about 30 per cent. more. Steel for tension members should not contain more than .26 per cent. of carbon, nor more than .1 per cent. of phosphorus. For compression members a harder steel or one containing from .34 to .42 per cent. of carbon is used in the Henderson bridge. A sample $\frac{3}{4}$ inch diameter, rolled from an ingot sent from each charge, was required to bend to 180° round its own diameter, to have an elastic limit of not less than 50,000 pounds, an ultimate strength of not less than 80,000, and an elongation of 15 per cent. in 8 inches, and a reduction of at least 30 per cent. at fracture and to be capable of tempering.

In designing steel members, it is very important that there should be no sharp angles or changes of section. There is no fibre in steel, and if a crack is found, however minute, it will increase until the piece breaks. It is on this account that the surface should be carefully examined to see that there is no scratch which could develop into a crack. Some excellent steel for eye bars assayed .16 per cent. carbon, .059 phosphorus and .66 manganese.

In steel eye bars the section across the eye does not need to be more than one-third more than in the body of the

bar, while for iron bars we have seen that one-half more at least was required.

When the contract for a bridge is let the railroad company appoints an inspector to see that it is properly made. He learns from the bridge company, which usually does not make the iron, where the orders for it have been placed, and then puts himself in communication with the iron firms, to know when the iron is to be rolled. He visits the works at that time and looks at the process. He may examine the way the iron is "piled" in the "fagots" of the blooming mill, but the manufacturers claim, with some show of justice, that if the iron bears the requisite tests, the inspector has no business with the method of manufacture. At all events, he has pieces cut off the ends of such finished bars as he chooses to indicate, which are shaped to a specified size for the testing machine. He has at least one such piece made from each section rolled. If angles $4 \times 3\frac{1}{2} \times \frac{5}{8}$ and $4 \times 3\frac{1}{2} \times \frac{1}{2}$ are rolled, he takes a test piece from each. So with plates, he may take one from a $12 \times \frac{5}{8}$ and a $12 \times \frac{1}{2}$ plate.

Test pieces of rods are generally the full size of the rod.

The dimensions of the test piece are first measured with calipers, which read, by estimation, to thousandths of an inch, and indentations are made in it at intervals of one inch apart for its whole length by striking a steel-pointed punch with a hammer, to be used in measuring the elongation. The piece is then broken in the testing machine. If this is worked too fast, the piece will not stand as much as if worked more slowly. The manufacturer's agent, however, who controls the speed, looks out to see that the stress is sufficiently gradually applied. He keeps balancing the pull applied at one end of the piece by weights applied to a series of weighing levers attached to the other end. When the load on the piece approaches the elastic limit, the machine is run very slowly. At a certain point

the material gives way suddenly and the weighing beam drops; this indicates the elastic limit. It may also be noted by measuring the increased distance between the punch marks. When this distance suddenly increases in rate, the elastic limit is passed. The machine is then run more rapidly until the piece breaks. The dimensions of the broken part are then measured and compared with the original area, and the reduction calculated as a percentage. For example, a piece cut from a $5 \times 3\frac{1}{2} \times \frac{5}{8}$ angle measured originally $.918 \times .625 = .574$ square inch. The beam of the machine dropped at 17,300 pounds, and the specimen broke at 28,450 pounds, with a reduced area of $.812 \times .508 = .412$ square inch. We have then the elastic limit at $\frac{17{,}300}{.574} = 30{,}100$ pounds per square inch; the ultimate strength at $\frac{28{,}450}{.574} = 49{,}500$ pounds per square inch, and the reduction $\frac{.574 - .412}{.574} = 28.2$ per cent. The elongation in 8 inches was 1.82 inches, which was equal to 22.75 per cent. The fracture was fibrous, of a uniform tint and good. As the specification required that it should stand 46,000 pounds per square inch, with an elastic limit of 23,000 pounds and an ultimate stretch of 15 per cent. in 8 inches, the test showed a satisfactory quality. Another test piece that had been prepared at the same time was taken to a blacksmith, who bent it cold by striking on the ends with a hammer to a right angle, with a radius at the inside of less than $1\frac{1}{4}$ inch (twice its thickness). No cracks appearing, the iron was accepted. If the iron is "burnt" or bad, it may sometimes stand the ultimate tension test, but will probably fail in the elongation. Its fracture will not then be of a uniform color, but will show bright spots, like crystals. These are the usual tests. Plate iron is also sometimes made to undergo the drifting

test. A hole is punched for a $\frac{3}{4}$-inch rivet, with the centre of the hole $1\frac{1}{2}$ inches from the edge of the plate. This is then enlarged with a steel drifting pin to 1 inch without showing fracture.

After the tests are satisfactory, the material is all looked over by the inspector, piece by piece, men turning it for him, so that he can see both sides. If there are nicks in the edges, which may indicate "burned" or "hot short" iron, if he thinks they do so, or they are too deep in size, he condemns the piece. If he accepts it, he marks it with his hammer, which has a die cut on its face of his personal mark. This is to prevent another piece being substituted. When the iron is not manufactured at the bridge works, but is shipped to them by rail, this surface inspection is generally made when the pieces are being loaded for shipment.

The tests on bridge steel are similar. Its fracture does not show the fibrous structure of iron, but is of a white, silky appearance, with 30 to 40 per cent. reduction of area, and a cup-like fracture, one end being rounded and fitting into a cup at the other end. If hard steel is used, it is only for parts in compression. If very soft steel, it is only for rivets, and as the latter cannot well be annealed after driving, and may be injured in this process, many engineers specify iron rivets even in steel members.

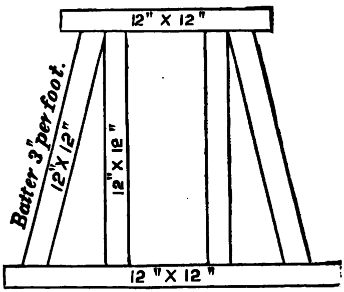
As steel is so liable to injury in working, it is usual to test first a piece from each ingot (which weighs probably 7 tons), and also a piece from each furnace charge, cut from the finished piece. Each ingot has its number stamped upon it, and each piece has the ingot number and also the number of the furnace and charge in addition, so that the pieces that are made at the same time can be easily picked out if a test shows that the steel has been spoiled in the converter or in the reheating furnace. The test piece from the ingot is generally a round rod of $\frac{3}{4}$ inch diameter.

The surface inspection of steel should be more minute than that of iron. A very fine crack should condemn it. Each ingot is closely examined in the mill by the manufacturer, and all such defects are cut out with a chisel, before the ingot goes to the furnace to be rolled and cut into what might be called blooms, or the pieces that are charged into the reheating furnaces, from which the finished bars or plates are rolled.

After the iron or steel goes to the bridge works, the inspector follows, and all parts of the works are open to him, so that at any time he can examine the process and object to anything that he thinks will affect the strength or appearance of the bridge. He tests all dimensions; the sections he should already have tested by measurement at the rolling mill; and at the bridge works he verifies the lengths of the finished pieces, the distance apart of the rivets, the closeness of fit of the details, etc.; also the straightness of the pieces. Steel is always annealed at the bridge works if it has undergone any reheating for making eye-bar heads. If any punching has been done on it, the holes should always be reamed out, so as to remove a small ring of metal that is always injured by the punch. In annealing, the furnace must not be too hot nor the bars suddenly cooled.

Rivets are tested after being driven by placing two fingers of one hand on one head and striking the other two sharp blows with a hammer, one to the right and the other to the left. If it feels loose, it is chalked to be cut out and replaced. The heads should be well made, especially not bent on one side.

TRESTLE WORK.



In countries where stone cannot be found, the railroad is built without culverts, being carried over streams on pile trestles. In countries where stone exists, wooden trestles often carry the railroad over valleys which it will take too long to fill up with earthwork, or where it may be too expensive to borrow it, at least before the building of the track enables it to be brought from a distance. In both cases the trestles are looked upon as temporary expedients, to be filled in with culverts and embankments, when their cost will be reduced by facility of transporting materials. Sometimes a ravine can be more economically spanned by an iron trestlework than by an embankment, which is then permanent.

Wooden trestles are either formed of piles, braced, or of squared timber framed into bents and supported on either piles, mud sills, or masonry. The bents are generally placed 16 feet apart. Pile trestle bents consist of four piles, the middle ones about 5 feet apart, and the outside ones about 3 feet apart. Up to 6 to 9 feet high they are not braced, but where the height exceeds this they are braced with two pieces, one on each side, of 3×12 stuff, spiked with three $\frac{5}{8}$-inch cut spikes, 8 inches long, to each pile and to the cap, the latter being formed of a

12 × 12 stick, 14 feet long, bolted to the piles with ¾-inch drift bolts, 21 inches long.

When the height exceeds 16 to 20 feet, it is divided into two stories by horizontal braces of 3 × 12 scantling, and longitudinal braces of 4 × 12 connect the bents at the top of the first story, one bolted on each side. Both stories in a bent are also braced transversely by 3 × 12 or 6 × 8 inch stuff. The two outer piles are often battered 1 or 2 inches per foot.

When the height is between 20 and 30 feet, the bent is made of five or six piles, 3 feet apart, outer ones battered 1 or 2 inches per foot.

Where the bent is formed of square timber, the scantling is all of 12 × 12 inches, braced with 3 × 12. There are two plumb sticks 5 feet apart and two inclined at a batter of 2 or 3 inches per foot, either spaced 2½ feet from the others at the top or touching them. These bents are divided into two stories when above 20 feet, and into an additional story for each additional 10 feet of height, the stories, however, being of equal heights. The posts are connected to the sills, and to each other when they are not long enough to make the whole length, by ¾-inch iron dowel pins, 8 inches long. The cap is drift-bolted to the legs, and each story is longitudinally braced with the adjoining ones with a horizontal piece 6 × 8 bolted to the vertical legs.

On top three stringers rest, for each rail. They are of 8 × 16 or 9 × 14 inches, spaced 2 to 4 inches apart by packing pieces 4 feet long at each bent, bolted together with ¾-inch bolts. They are bolted to the cap with ¾-inch drift bolts, 21 inches long.

If the bent of square timber rests on piles, one of the latter is driven under each leg of the trestle and cut off about 2 feet above the ground. If it rests on mud sills, these are of 12 × 12 inches, 4 feet long, placed at right angles to the sill under each post, and at enough interme-

diate points to get an equal amount of bearing surface to that in the sill. The sill of the bent is bolted to each with a $\frac{3}{4}$-inch drift bolt, 21 inches long.

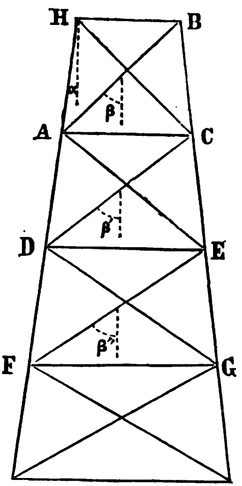
Into the stringers 6×8-inch oak ties, 10 to 12 feet long, are notched half an inch. This keeps the ties from bunching together, as they will otherwise do under the stress of traffic. They are spiked to a stringer with $\frac{1}{2}$-inch spikes, 10 inches long. When a guard rail is used, it is notched on the ties, which then need not be notched in the stringers, but the guard rail is spiked to every other tie, alternating on the two sides. Where stone is plenty, the sills should rest on masonry piers, 2 feet wide.

When a mortise and tenon are used, the width of the tenon may be made from one-third to one-fourth the width of the stick. A tree-nail passes through both mortise and tenon, to hold them fast. It is usual to bore the holes for this, not directly opposite to one another, but that in the tenon a little nearer the shoulder than would be opposite that in the mortise. This will make the driving of the pin draw the post tightly against the cap. In a trestle designed by the writer for carrying water-pipe, a tenon 5 inches long on a 6×8 stick, in a cap of the same size, with a pin $1\frac{1}{2}$ inch in diameter, was made to draw $\frac{1}{8}$ of an inch, which proved to be a very little too much, as it cracked some of the caps; not enough, however, to destroy any or make them unfit for their duty. However, a draw of $\frac{3}{32}$ would have been better for this size of timber. One-eighth draw would do for 12×12 sticks.

High trestle-work is generally made of iron. It is usually made to batter in one direction, and it is better that the batter should be in a plane at right angles to the centre line of the bridge. Abroad it is often made to batter in a plane parallel to the centre line, but as the force due to the wind acting to overturn it is greater, when the structure is very high, than the reaction due to the force on the driv-

ing wheel, the additional strength due to the greater leverage obtained by spreading the base should be opposed to the wind pressure. The high piers built by American engineers usually batter at one and one-half inch to a foot, but the columns at opposite sides of the bridge are in vertical planes at right angles to it. The following formulas represent the stresses in the several parts of such a pier in one of these vertical planes.

Let $P =$ the pressure due to the wind on the train, and trusses on one pair of columns, found by multiplying the side surface in square feet of a train covering half of each adjacent span and the exposed surface of the half spans themselves by the maximum pressure of the wind on a square foot. Let $P' =$ the pressure due to the wind



on one panel of the pier, on one pair of columns, found by multiplying one-half the surface exposed on the adjacent sides of the bent, by the maximum pressure of the wind.

Let $W =$ the vertical load due to train and bridge on one column; for the outer columns it will be one-half the load on the adjacent spans of bridge and pier.

Let $W' =$ the vertical load due to one panel of the pier on one column. Let $\alpha =$ the angle made by the columns with the vertical. The batter is usually, in American structures, 1½ inches to a foot, when α will $= 7° \; 8'$.

Let β, β', β'', etc., $=$ the successive angles which the diagonal braces make with the vertical, from the top downwards.

Then,

Stress on $HB = P + W \tan. \alpha + \frac{1}{2} W' \tan. \alpha$ compression.

Stress on $AC = P \dfrac{\sin. (\beta - \alpha)}{\sin. (\beta + \alpha)} + P' + W' \tan. \alpha$ compression.

Stress on

$$DE = P \frac{\sin. (\beta - \alpha) \sin. (\beta' - \alpha)}{\sin. (\beta + \alpha) \sin. (\beta' + \alpha)} + P' \frac{\sin. (\beta' - \alpha)}{\sin. (\beta' + \alpha)} + P' + W' \tan. \alpha.$$

Stress on

$$FG = P \frac{\sin. (\beta - \alpha) \sin. (\beta' - \alpha) \sin. (\beta'' - \alpha)}{\sin. (\beta + \alpha) \sin. (\beta' + \alpha) \sin. (\beta'' + \alpha)} + P' \frac{\sin. (\beta' - \alpha)}{\sin. (\beta' + \alpha)} \frac{\sin. (\beta'' - \alpha)}{\sin. (\beta'' + \alpha)} + P' \frac{\sin. (\beta'' - \alpha)}{\sin. (\beta'' + \alpha)} + P' + W' \tan. \alpha$$

&c. = &c.

Stress on $AB = P \dfrac{\cos. \alpha}{\sin. (\beta + \alpha)}$ tension.

Stress on $DC = P \dfrac{\cos. \alpha \sin. (\beta - \alpha)}{\sin. (\beta + \alpha) \sin. (\beta' + \alpha)} + P' \dfrac{\cos. \alpha}{\sin. (\beta' + \alpha)}$ tension.

Stress on

$$FE = P \frac{\cos. \alpha \sin. (\beta - \alpha) \sin. (\beta' - \alpha)}{\sin. (\beta + \alpha) \sin. (\beta' + \alpha) \sin. (\beta'' + \alpha)} + P' \frac{\cos. \alpha \sin. (\beta' - \alpha)}{\sin. (\beta' + \alpha) \sin. (\beta'' + \alpha)} + P' \frac{\cos. \alpha}{\sin. (\beta'' + \alpha)}$$ &c. = &c.

Stress on $AH = P\dfrac{\cos.\ \beta}{\sin.\ (\beta + \alpha)} + \dfrac{W}{\cos.\ \alpha}$ compression.

Stress on

$$DA = P\frac{\cos.\ \beta}{\sin.\ (\beta + \alpha)} + P\frac{\cos.\ \beta \sin.\ (\beta - \alpha)}{\sin.\ (\beta + \alpha) \sin.\ (\beta' + \alpha)} +$$

$$P' \frac{\cos.\ \beta}{\sin.\ (\beta' + \alpha)} + \frac{W}{\cos.\ \alpha} + \frac{W'}{\cos\ \alpha} \text{ compression.}$$

Stress on $FD = P\dfrac{\cos\ \beta}{\sin\ (\beta + \alpha)} + P\dfrac{\cos\ \beta \sin\ (\beta - \alpha)}{\sin\ (\beta + \alpha) \sin\ (\beta' + \alpha)}$

$$+ P\frac{\cos \beta \sin (\beta - \alpha) \sin (\beta' - \alpha)}{\sin (\beta + \alpha) \sin (\beta' + \alpha) \sin (\beta'' + \alpha)} + P'\frac{\sin (\beta' - \alpha) \cos \beta}{\sin (\beta'' + \alpha) \sin (\beta' + \alpha)}$$

$$+ P'\frac{\cos\ \beta}{\sin\ (\beta'' + \alpha)} + P'\frac{\cos\ \beta}{\sin\ (\beta' + \alpha)} + \frac{W + 2\ W'}{\cos\ \alpha} \text{ compression.}$$

Stress on $A\ H$ = no tension.

Stress on $D\ A = \dfrac{W}{\cos \alpha} + \dfrac{W'}{\cos \alpha} - P\dfrac{\cos \beta}{\sin (\beta + \alpha)}$ tension, if a minus quantity; if it is a plus quantity, there is no tension on $D\ A$.

Stress on $F\ D = \dfrac{W}{\cos \alpha} + \dfrac{2W'}{\cos \alpha} - P\dfrac{\cos \beta}{\sin (\beta + \alpha)} - P\dfrac{\cos \beta' \sin (\beta - \alpha)}{\sin(\beta + \alpha) \sin(\beta' + \alpha)}$

$-P'\dfrac{\cos \beta'}{\sin (\beta' + \alpha)}$ tension, if a minus quantity.

Make these calculations for the bridge loaded with its greatest load, and also unloaded; the pressure of the wind per square foot being taken the same in both cases. The tension on the lowest columns in the unloaded bridge (if any), due to the differences in the stresses produced by the wind and load, may exceed that produced under the same circumstances with the loaded bridge.

Prof. Wm. H. Burr has called attention to the fact that the effect of the wind on the train is to raise the windward side and so increase the load on the leeward side, decreasing and increasing the loads on the respective columns as

given above. Similarly the effect of the wind on the trusses when the top chord rests on the pier is to decrease the load on the leeward side, and increase it on the windward. The effect produced by the difference of these should therefore be subtracted or added (as the moment on the train or truss is the greater) to that produced by P above. This may be accomplished by writing $P \pm 2\, t \tan. \alpha$ for P and $W + t$ for W for the leeward, and $W - t$ for the windward column, in the formulas above, t being equal to

$$\frac{H\,h - H'\,h'}{l}$$

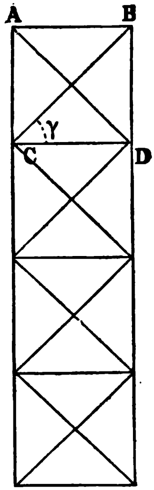
in which H is the pressure of the wind on the train covering the adjacent half spans, and h is the height of the centre of pressure above the point of connection of the truss with the piers, and H' and h' are similarly the total pressure of the wind on the half span and the depth of the centre of pressure below the point of connection of the truss with the pier, and l is the distance $H\,B$ in the figure on page 128.

In a direction parallel to the centre line of the bridge, the pressure on the braces is due to the action of the driving-wheel of the locomotive, and is found as follows:

Let $\gamma =$ the angle made by the brace $B\,C$ with the horizontal. If T is the horizontal force exerted by the steam in the cylinder, and S is the stroke of the piston, and D is the diameter of the driving-wheel, the stress on $A\,B$, $C\,D$, &c., will be $T\frac{S}{D}$, and that on $A\,D$, &c., equal to $\frac{S\,T}{D \cos. \gamma}$

To find T, if the cylinder of the locomotive is d inches

A B



in diameter, and the pressure of the steam in the boiler 140 pounds per square inch,

$$T = 140\pi\frac{d^2}{4} = 109\, d^2$$

The value of $\frac{S}{D}$ is about two-fifths in the heaviest engines on the ordinary gauge.

The stress on $A\ D$ resolved on $A\ C$ must be added to the compression due to the weight of the train and bridge, and the effect of the wind, on the same piece. The next column below will have twice this amount to be added to the compression due to the weight of the train and bridge and the effect of the wind, the next three times, and so on.

The pressure in the steam cylinder is never as great as that in the boiler, and will likewise be less the faster the engine is running. If the engine, however, moves slowly from a state of rest, the stresses will approach those given above.

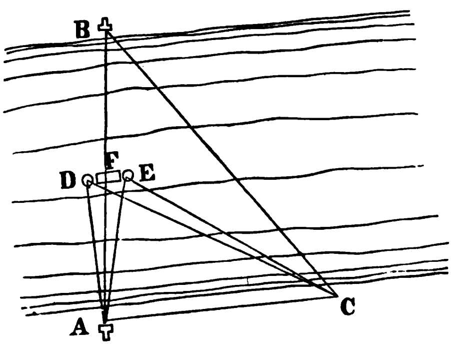
A correspondent of *Engineering*, Vol. 4, p. 454, has shown that in a viaduct the most economical arrangement of the spans as regards quantity of material occurs approximately when each span costs the same as its adjacent pier. In an iron viaduct this will be nearly the case when the span is equal to one-half the height of its adjacent pier. If the unevenness of the ground causes a great variation in the heights of different piers, the advantage of uniformity in manufacture would perhaps cause the adoption of a span of one-half the average height of all the piers.

MEASUREMENTS OF BRIDGE SPANS.

As the young engineer may be called upon to stake out the masonry of bridge piers and abutments when there may be some difficulty in making a direct measurement at the place where the bridge is to be, descriptions will be given of the methods which have been employed in several actual cases of the kind, which presented some interesting problems.

First, the Mahoning Creek bridge, on the Allegheny Valley Railroad, in Pennsylvania.

This consisted of two spans each of 150 feet. Plugs

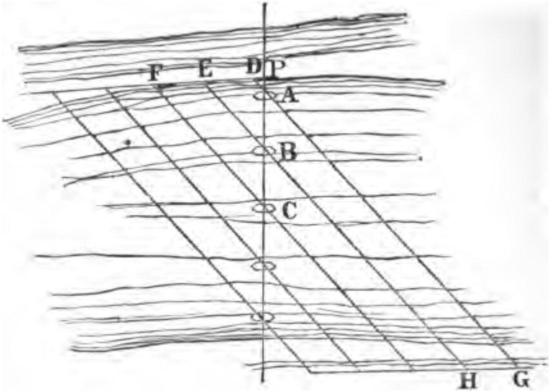


were put in on the centre line at A and B, and a base line measured on the sandy shore $A\ C$, and the angle $B\ A\ C$ likewise measured. The distance $A\ B$ was then calculated, and the position of one of the abutments decided upon. The distance from this abutment to the centre of the pier was then, of course, known, and points D and E were assumed at known distances from the centre of the pier, in its centre line produced. The water was sufficiently shallow to allow long iron rods to be driven into the bed of the creek and project above the surface at low water. The angles $D\ A\ C$ and $D\ C\ A$ were calculated so that with a transit at both A and C with these angles turned off, the rodman, in a boat, could be directed one way or another until the iron rod was in line from both instruments; it was then driven down. Similarly a rod was driven at E. For calculating the angle $D\ A\ C$, the angles $D\ A\ F$ $\left(= tan.^{-1}\ \frac{D\ F}{A\ F}\right)$ and $F\ A\ C$, (measured,) were added together. From $A\ D \left(= \sqrt{\overline{D\ F}^2 + \overline{A\ F}^2}\right)$ and $A\ C$, (measured,) and the included angle, calculated above, the angle $D\ C\ A$ was calculated. Similarly the angles to the point E were calculated. As the engineers' camp contained but one transit, the level was used in lieu of another. The transit having been set at A and the angle $D\ A\ C$ turned off, a plug was put in on the opposite side of the creek; the level was then set over the point A and sighted on the plug and clamped. The transit was then set at C and the angle $A\ C\ D$ turned off; by its means and the level the rod was set at D. This method, which was devised by the principal assistant engineer, though ingenious, was not perfectly satisfactory to the engineer in charge. The iron rods were used, however, for placing the wooden platform on which the masonry was built.

Spikes were driven in its centre line at each end, and it was floated until they were at the proper distance from the rods. The masonry was then built, and settled the platform on the bottom. Just below the water-line an offset of six inches was left all round, the neat work beginning at that distance from the edge. The want of confidence in the method was principally because of the inferior character of the transit employed, which, to secure lightness, had a small graduated limb, and had seen some rough service. Accordingly, when the neat work was to be started, a stone was laid on the foundation to raise it out of the water, and a line was stretched from it to the abutment. The assistant then took his rodman in a boat, with a rod ten feet long, which was applied successively to the line, beginning at the abutment, a common pin being thrust through the line at the front end of the rod, to which the hind end was then held. This measurement was not in a straight line, because the string sagged. This was corrected by using inversely Weisbach's formula for the length of a suspension-bridge cable, having the span and "versin." $L = \frac{2}{3} \frac{V^2}{S}$; the versin. being estimated by holding a level rod at the middle and sighting from one end to the other, observing where the visual line cut the rod. This second measurement, by which the bridge was built, differed about one foot from the triangulated distance, which was allowed for in the framing of the bridge.

The next example is that of the Delaware River bridge, on the North Pennsylvania Railroad route to New York. It was devised by Mr. D. McN. Stauffer, the engineer in charge.

He took advantage of the cold winter weather, when the river was frozen over, and measured the spans carefully on the ice, marking the different centres of the piers A, B, C,

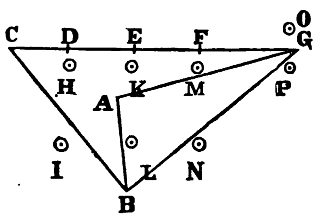


etc. He then put iron plugs, one inch in diameter, four feet long, on the shore at D, E, F, etc., in line, and at equal distances apart. He set his transit over D, sighted on A, and put in a plug on the opposite side of the river at G; similarly H was put in on the line $E\,B$ produced; etc. The piers were built on wooden platforms, which were floated into place when the ice had disappeared. The centre line having been drawn on each platform, and marked by a long pin on each side, and a short one with red flannel attached in the centre, a transit at P served to put it in proper line, while one at D, E, F, etc., successively served to correctly place the centres. An unforeseen difficulty arose from boys stealing some of the iron plugs, but the ground being frozen the holes remained, and they were easily replaced.

The final example is that pursued by Mr. L. L. Buck for the Verrugas Viaduct, on the Callao, Lima and Oroya Railroad, in Peru. This viaduct is situated in some of the wildest scenery of the Cordilleras, at a height of nearly 6,000 feet above the sea. The slopes of the mountains are exceedingly steep, generally too much so to permit of ascending until a road is cut. There are often places,

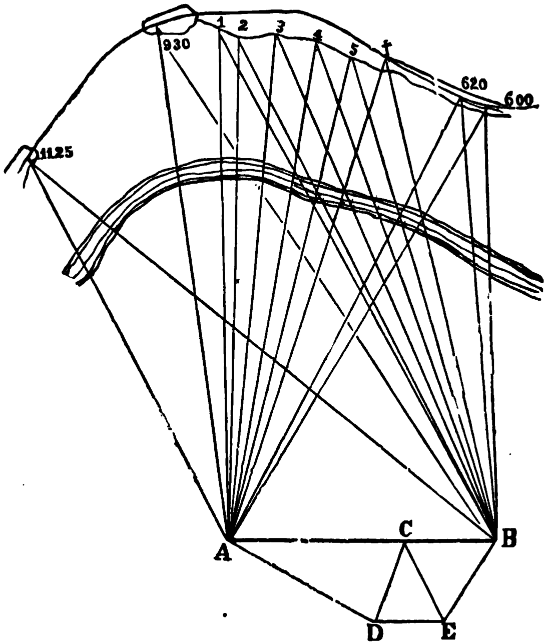
however, on the tops of small hills, or on their sides, where the slopes are less abrupt, and these were taken advantage of by the ancient inhabitants of the land for terracing in successive level steps, over which the water, which is necessary for irrigation, was conducted by canals. The climate is very dry, the rainy season being confined to six months, and even then consisting of merely light rains in the afternoon. There being no frost the hills retain the steep character of the volcanic disturbances which created them. The old Inca terraces, however, formed admirable places for measuring a base line.

The Verrugas viaduct consists of one span of 125 feet, and three spans of 100 feet, supported by abutments, and by three iron piers of heights of 177, 252 and 152 feet respectively, each pier measuring 50 feet across on the centre line of the railroad. The slopes were too abrupt on the sides to permit of descending on the centre line of the bridge.



A base line A B was accordingly measured in the valley, 197 feet long, the positions of the piers being D, E and F. Points C and G were put in on the centre line near the faces of the abutments. The angles from A B to G were observed, and the distance B G calculated; from this distance, with observed angles at B and G, C G was calculated. Another base was then measured on a level terrace above, 231 feet long, but badly situated with reference to C G. By a series of triangulations C G was again obtained. It differed from the first value by four-tenths of a

foot. The first triangulation was supposed to be the most accurate, and stakes were put in from it at *D*, *E* and *F*, the centres of the piers, and guarded by turning off right angles, and putting plugs at *H*, *I*, *K*, *L*, *M*, *N*, *O*, *P*. As the bridges were made complete in the United States, and shipped to their destination, it was necessary that they should come together exactly on erection. Accordingly the following check was made on the above measurements and calculations: A line was measured on a level terrace, putting stakes every fifty feet, which were driven to the same level, and measured with a tape and spring balance. Stakes were also driven twenty feet apart, and the distance measured with a rod twenty feet six inches long, made deeper in the middle and tapering towards the ends, so as not to bend with its own weight. Marks were made on this rod exactly twenty feet apart. This latter measurement agreed with that of the tape and balance. Stakes were put in on this line at the points corresponding with the centre of each pier and the faces of the abutments. A long flat wire ("hoop-skirt wire") was then suspended over this base line, with one end fast and the other passing over a pulley, with a weight hung to the latter end. The centres of the piers and faces of the abutments were then "plumbed up" from the stakes, and marked, by winding, first with shoe-thread waxed, and then with white thread. The wire was afterwards suspended over the place which the bridge was to occupy, stretched with the same weight as before, and sights taken on it from the stakes at *I*, *L*, *N* and *O*, using *H*, *K*, *M* and *P* as fore-sights. It was extremely satisfactory to Mr. Buck to find that one of the marks on the wire only varied one-half an inch, and all the others less than three-eighths of an inch, from the triangulated distances. For measuring spans, steel pianoforte wire No. 10, stretched with 15 pounds tension, is recommended by Col. Patton.



As a very interesting example of triangulation, the following, executed by Mr. O. F. Nichols, may be described:

It was necessary to locate the Callao, Lima & Oroya Railroad over a very precipitous cliff, which was itself inaccessible. The line was run from each direction as far as possible, to stations 620 and 1125 in the figure. At 930 a point about grade could also be approached, and a small place was graded there, and fixed as a portion of the line. The relative positions of these points were determined by triangulation from the opposite side of the valley. They were then plotted with the line as determined on each side of 1125 and 620, and connected on paper with proper lines, and tunnels driven to connect them. In locating these lines on paper, it was borne in mind that the tunnels

should always continue in the rock, and not cross any seams that might run in from the face of the cliff. Points on this face were therefore likewise determined by triangulation and plotted. On the side of the valley from which the triangulations were made, there existed numerous Indian level terraces, made for purposes of irrigation, which were utilized for the base lines. The points *A* and *B* were fixed upon at convenient places, and an intermediate point *C* marked. A base line *D E* was then measured very carefully several times. From it the distances *A C* and *C B* were calculated, and added together, giving the distance *A B* as a longer base line. From *A* and *B* the successive points 620, 930, 1125, 1, 2, 3, 4, 5, †, etc., were observed; some of these consisting of discolorations of the rocks, produced by growing lichens, and others of marks made by firing rifle balls against the cliff near the grade line. Having afterwards calculated the various distances and angles, and plotted them, the line was fixed. From 930 to 1125 an interesting problem arose. The direction of the tangent at 930 with the lines to the extremities of the base line was fixed, 930 was taken as the beginning of the curve with a given radius, and the length of the curve was so calculated that a tangent at the other extremity should pass through 1125. The tunnel was driven from both ends and the headings met on the tangent. It was therefore necessary to run the curve to the tangent point very accurately, and the angle turned off should coincide with the centre line run from the other end. As an example of Mr. Nichols' accurate work under great difficulties, it may be mentioned that when the headings met there was an error of less than two inches, the tunnel being 537 feet long, the base line 886.79 feet long, and situated 1750 feet off, the work prosecuted under a tropical sun, in a country so malarious that the men died, during the sickly season, at an average of about thirteen

a day. Between 930 and 3, similar calculations were made and a tunnel driven. In addition to the lines shown in the figure, two other base lines were calculated and measured as bases of verification; they are omitted, however, as they would too much complicate the figure.

MASONRY.

In masonry, the two important points required are to have good beds to the stones, and to have a good bond. When not closely watched, masons working for contractors will put "nigger-heads" into the wall, that is, stones from which the natural rounded surface of the stone or boulder has not been taken off. Each course should be made level before beginning the next one. There should be plenty of headers, to run into the wall as far as possible. A trick of the masons is to use "blind headers," or short stones that look like headers on the outside, but do not go deeper into the wall than the adjacent stretchers. When a course has been put on top they are completely covered up, and, if not suspected, the fraud will never be discovered unless the wall shows its weakness. The headers may taper off at the back in their width, but should retain their depth throughout. Rankine recommends that one-fourth the face of a wall should consist of headers, whose length should be from three to five times the depth of a course. Some engineers prefer masonry laid dry, as it requires a superior class of work, and imperfections are more easily detected. For railroads, however, it is apt to be shaken loose by the motions of the trains.

Mortar is made of a mixture of 1½ bushels of lime to 3 of sand, the lime to be slacked (one part water to one of lime) immediately before using, before the sand is put in; the sand to be sharp, and better if the grains are large (coarse). This amount will lay a perch of stone. Vicat says the best proportions are 2.4 of sand to 1 of lime

(Rankine). If cement is used, not too much water should be mixed with it. It is found that the cement becomes hardest if just enough water is mixed to make it moderately dry, but capable of being finished and smoothed off with a trowel. The proportions are 1 of water to 4 of cement, by measure. Cement used neat is liable to crack. For this reason, and the sake of economy, it is mixed with an equal measure of sand, the larger and sharper the better. Above the water line, 2 parts of sand to 1 of cement may be used. For mixing cement and sand, an old wine barrel is better than a mortar box. In the latter it is difficult to prevent the sand from floating on top; that is, it is more difficult to get them thoroughly mixed.

It used to be the custom to lay about four inches from the face in cement mortar thick, and to grout the remainder with the same mortar, with its consistency so reduced by water that it would run into the interstices. As it is found, however, that this excess of water injures the quality of the cement and the sand tends to separate from the cement, the practice is being given up. When a a coating of cement is spread over a surface, it should be rubbed with a flat, smooth piece of wood, technically called a "float," until it begins to set; otherwise it will crack. For making a cubic yard of concrete, a cubic yard of stones, broken to pass through a 2-inch ring, is mixed with the cement mortar previously made with one barrel of cement and two barrels of sand, with about one-third to one-half a barrel of water. It is consolidated by throwing from a height or ramming until the water begins to come to the top. When it is in place it should be allowed 24 hours to set before covering it with water. Some American cements may require double the proportion of water to that given above. For testing cement, it is recommended by Mr. Faija that it should be fine enough to pass entirely through a sieve having 25 meshes per

lineal inch, and should not have more than 10 per cent. residue on a sieve of 50 meshes; that briquettes should stand a tensile stress of 175 pounds per square inch after three days and 350 pounds after seven days.

FOUNDATIONS.

The best foundation is rock. Rankine says the crushing strain of limestone and sandstone is from 3,000 to 8,500 pounds per square inch, and granite 10,000 to 13,000. He says the actual pressure should in no case exceed one-eighth the crushing strain. He likewise says that foundations in gravel, hard clay, and sand are usually loaded with from 2,500 to 3,500 pounds per square foot. At Nantes, 17,000 on sand caused some settlement. The anchorage of the Brooklyn Bridge is built on a wooden platform, three feet thick, which rests on sand 22 feet below the surface, with a pressure of four tons per square foot. (Trans. Amer. Soc. of Civil Engineers, Aug., 1874.) Mr. H. Leonard, in *Engineering*, Vol. 20, p. 103, says that, from the result of actual experiments in India, alluvial soil will safely bear 2,240 pounds to the square foot. He says the depth of such foundations should not be less than four nor more than eight feet. The offsets of the foundation should spread at an angle of not more than 45 degrees, and no step should be less than 18 inches high: if less, it may break off. Sir Charles Fox says his experiments show that alluvial soil will bear 1,680 pounds per square foot. Rankine says the usual rule for spreading the foundation of a wall is to make the breadth of the base 1½ times the thickness of the body of the wall in compact gravel, and twice the thickness in sand and stiff clay. Although brick alone will stand a crushing stress of from 3,000 to 15,000 pounds per square inch; when it is built up in a pier with mortar, the structure will sometimes begin to show signs of failure at only 500 pounds per square inch. (See mechanical

tests of building materials made by the Public Buildings Commission, Philadelphia, 1884.)

The foundation, especially in sand and alluvial soil, should be kept from the effects of running water. This may sometimes be avoided by "rip-rapping" round the foundation.

A common way of building on a gravel foundation in streams where the water does not exceed five feet in depth, is to build on two courses of 12 × 12 inches squared timber, laid close, at right angles to each other, and spiked at each intersection. This is floated in place, and the masonry built until it sinks. For deeper water, there would be a risk of upsetting before reaching the bottom, and guide piles must be driven on the outside. Foundations on gravel or rock, in water, are often built on cribs. A framework, the size of the crib, should be floated to the proper place, and soundings taken all round it at intervals of about three feet, with some intermediate ones across. The bottom of the crib is then shaped to the surface given by the soundings, and so will have an even bearing. The crib is made in open cells, about 6 feet square, with a floor at the bottom to hold the loose stones which are filled in to sink it. The outside is made with 12 × 12 squared sticks, fitting close, laid horizontally. Where the sticks forming the cells come through the sides, they are dovetailed to them, and spiked besides. All these side sticks are spiked together with one inch spikes which are long enough to go through three of them, so that each one is spiked to the two below.

The sides of the crib are generally made with a batter of 1½ inch to a foot.

When the bottom is a soft mud, piles must be resorted to for a foundation. They are generally driven about 2½ feet apart, sawed off at least two feet below the low water

mark, and capped with 12 × 12-inch sticks. On these a floor of the same sized timbers is laid at right angles to the lower course and close together, and on the latter the masonry is built.

PILE DRIVING.

Weisbach's formula:

$$L' = W\left(\frac{W}{W+W'}\right)\frac{h}{d}$$

where W = the weight of the ram in pounds.
W' = the weight of the pile in pounds.
d = the depth which the last stroke drives the pile in inches.
h = the height of fall in inches.
L' = the load which the pile will just bear in pounds.

The Dutch engineers use a similar formula, except that they use for d the average penetration per blow, got by taking the whole penetration in, say, 100 blows and dividing by 100. They also use a factor of safety of $\frac{1}{8}$.

If W' is supposed to be so small in comparison with W that it may be neglected, and a factor of safety of $\frac{1}{8}$ is taken, we have Sanders' formula:

$$L = \frac{W}{8}\frac{h}{d}$$

where L is the safe load the pile will bear.

Rankine's formula, supposing the pile to be sustained by the friction on the sides, is

$$L' = \sqrt{\frac{4\,E\,S\,W\,h}{l} + \frac{4\,E^2\,S^2\,d^2}{l^2}} - \frac{2\,E\,S\,d}{l}$$

where E = the modulus of elasticity of the pile.
S = the sectional area of the pile in inches.
l = the length of the pile in inches.

We may take $E = 1{,}440{,}000$.

$S = 1\frac{1}{2}$ square feet $= 216$ square inches for an average.

$l = 30$ feet $= 360$ inches for an average.

Then the formula reduces to

$$L' = 1{,}859\sqrt{W h + 864{,}000\ d^2} - 1{,}728{,}000\ d.$$

Rankine recommends a factor of safety of 5, which will reduce the equation to

$$L = 372\sqrt{W h + 864{,}000\ d^2} - 345{,}600\ d.$$

Trautwine gives (reducing the value of his letters to the same measure):

$$L' = \frac{27\ W\sqrt[3]{h}}{1 + d} \text{ for the extreme load, and}$$

$$L = \frac{9\ W\sqrt[3]{h}}{1 + d} \text{ for the safe load.}$$

Rankine states that, according to the best authorities, the piles should be driven until $d = \frac{1}{10}$ of an inch, while Trautwine says the French consider it sufficient for d to equal $\frac{1}{15}$ of an inch, d being obtained in each case by dividing the total penetration in the last thirty blows by thirty. Many engineers, however, are satisfied to specify that the last blow with a 2,000 pounds hammer, falling 30 feet, shall not drive the pile more than half an inch. These values of d are only when the pile reaches a firm stratum. When it is entirely in mud, the driving may be stopped when the last blow drives it from one to two feet, a little time consolidating the mud about it. In such a case the pile should not be less than 60 feet long. Rankine's formula takes a simpler form when h is expressed in feet.

Rnnkine's formula, when h is in feet, and $d = \frac{1}{150}$, becomes

$$L' = 80 \left\{ 80 \sqrt{W h} - 144 \right\} \text{ for ultimate load, and}$$

$$L = 16 \left\{ 80 \sqrt{W h} - 144 \right\} \text{ for safe load.}$$

If W is expressed in tons, an approximate formula is

$$L' = 135 \sqrt{W h} \text{ for ultimate load, and}$$

$$L = 27 \sqrt{W h} \text{ for safe load.}$$

Rankine gives, as the limit of safe load on piles which reach firm ground, 1,000 pounds per square inch of head, and for the safe load on those which rely only on the friction of the mud against the sides, 200 pounds per square inch of head. Some actual experiments by Mr. E. T. D. Myers, on piles resting on a liquid mud, driven 45 feet deep, made nineteen hours after driving, gave 62 pounds only as the bearing power per square inch of head, this resistance, however, increasing with time. Some experiments in England (Van Nostrand's Magazine, Vol. 25, p. 275,) give 440 pounds per square inch of head as the withdrawing power of piles, or 1,875 pounds per square foot of contact for piles in stiff blue clay. For carrying railroads over marshes bents of four piles are usually driven 12 feet apart.

Rankine says the diameter should never be less than $\frac{1}{20}$ of the length. This probably refers only to piles resting on a hard stratum. Some of the piles supporting the bridges in the Hackensack marshes are 70 feet long, but *not* 3½ feet in diameter. The piles of the bridges which carry the Philadelphia, Wilmington & Baltimore Railroad over the Gunpowder River never reached a solid bottom. They are very long, however, and have proved perfectly satisfactory. Rankine says the best material is elm, and

they should be driven butt downward. The ends should be sharpened to a point, whose length is $1\frac{1}{2}$ times the diameter. The piles of the South Street Bridge, Philadelphia, were not sharpened, but were cut off square, to increase the bearing surface. Col. Patton found that the method of driving piles butt downwards was only necessary in quicksand, and where the pile would sometimes shoot up 8 to 10 inches after a blow if driven point downwards. (*Engineering News*, April 4, 1885.) Driving butt downwards is objectionable on account of the less quantity of heart wood at the small end, increasing the decay, the small end is easier shattered, and the pile drives more slowly owing to its less inertia.

Piles are usually capped with a wrought iron ring $\frac{1}{2} \times 2\frac{1}{2}$. The diameter of the butt is generally from $\frac{1}{25}$ to $\frac{1}{40}$ of the length.

Piles should not be less than ten inches in diameter at the small end.

The bearing power of discs on iron piles in sand is five tons per square foot, according to Brunlees.

For driving piles through boulders and gravel, a heavy ram and small fali is the best. For example, a ram of 50 cwt. and a fall of 8 to 10 feet. For driving through sand, the blows should be delivered rapidly, so that the sand should not have time to compact itself about the pile in the intervals. A gunpowder pile driver is good for this purpose. McAlpine states that the surface friction in driving cast-iron cylinders 6 feet in diameter, through rocky gravel, was one-half a ton per square foot. General Smith found the friction of sand on an iron cylinder only $1\frac{32}{100}$ pounds per square inch. Gaudard gives the friction between cast-iron cylinders and gravel at 2 to 3 tons per square yard for small depths, and 4 to 5 tons per square yard for depths of 20 to 30 feet. He also says that piles at La Rochelle, in soft clay, can support 164 pounds per

square foot of lateral contact, and at Lorient, in silt, 123 pounds. According to some observations made by Mr. Stauffer, at the South Street Bridge, in Philadelphia, the frictional resistance of mud on cast-iron cylinders was only 46½ pounds per square foot of surface. Mr. Andrews, in Cork Harbor, found the friction of mud on brick was 200 pounds per square foot. Col. Patton found that the resistance of sand mixed with boulders on the wooden caissons of the Havre-de-Grace bridge was from 285 to 489 pounds per square foot, the greater value depending on the greater number of boulders.

For driving piles in sand, a great deal of labor can be saved by applying a water jet to loosen the sand. With hollow iron piles, the water may be forced down the interior of the pile, and its own weight will carry it down rapidly. With wooden piles, a notch is made at the end of the pile, in which the nozzle of a 1½ inch diameter hose is inserted. Water is then forced through this nozzle.

When masonry is supported by piles, care should be taken that no horizontal thrust comes upon it, unless the piles are specially braced for it. On the Delaware Extension of the Pennsylvania Railroad there is a mass of masonry 38 feet long, 12 feet wide, and 24 feet high, which serves as the abutment to the bridge over the Schuylkill River. When, many years ago, the embankment was built behind it to about half its height, the abutment began to move outward. The embankment then had to be removed and replaced with trestles. The retaining walls of the Chestnut Street Bridge, in Philadelphia, were built on platforms resting on piles, a separate platform for each wall, the walls running back parallel to the centre line of the street. These platforms were not tied together. Some time after the completion of the work the walls began to show signs of being pushed over. Buttresses have been built to prevent their further movement, but it is feared

that it still continues, though certainly very slowly. If the platforms had been tied together at first by balks of timber, or otherwise, the thrusts of the walls would have been neutralized and their failure prevented.

The piles on which the "bulkheads" of the new docks at New York rest are braced together so as to prevent their being pushed out by the thrust of the material that they sustain behind them.

TRACK-LAYING

After the road is brought to sub-grade, the centre line is re-run, and stakes are set on each side of the road-bed at $4\frac{1}{2}$ feet off on a single-track road, and 100 feet apart, except on curves sharper than 3 degrees, where they should be 50 feet apart. These stakes are put one foot above the sub-grade, and give the top of the ballast. On curves, the outer one is elevated $\frac{1}{10}$ of a foot for each degree of curve above the inner one, which usually carries the grade. In the original construction it is better to depress the inner and elevate the outer, each $\frac{1}{20}$ of a foot, so that the centre of gravity may travel on the grade without being raised. This gives the elevation of the outer rail $\frac{1}{2}$ an inch for each degree of curve, which is what it ought to be, supposing a speed of $27\frac{1}{2}$ miles an hour. The rule for track-layers to have when there are no stakes set is, the elevation is equal to the middle ordinate to a chord of 48 feet of the curve. The theoretical rule for elevating the outer rail for 4 feet $8\frac{1}{2}$ inches gauge is:

$$\text{Elevation in inches} = \frac{v^2}{1{,}518} \times \text{degree of curve},$$

in which v is the velocity in miles per hour. This formula is derived on the supposition that the axles are always

radial to the curve, and it is sufficiently exact when the radius of the curve is not less than 1,000 feet. For curves of small radius, however, say of 200 feet, with engines having a rigid wheel base of 10 feet, the outer rail will be worn much more rapidly than the inner one, even though at a speed of 6 miles per hour it is elevated as much as 6 inches. The outer rail should never be elevated more than 8½ inches.

The track-layer places a wooden straight-edge, 8 inches wide, on the stakes at two consecutive stations, and has two pieces of wood, 8 inches long, held upright on a tie at the places where the rails come. The tie is then driven down until the visual plane across the straight-edges just touches the tops of the blocks. Having set three intermediate ties in this way, the remaining ones are set with a straight-edge 15 feet long, laid on two of the ties already set. All the ties having been set, the half-gauge is measured off at the stakes, and the rail spiked fast, the portion between two stakes being lined by eye. One line of rails having been spiked, the other is spiked with a gauge-rod applied at every tie.

For bending rails to a curve, they are allowed to fall on two supports, placed at some distance apart, and the stored-up energy due to gravity produces the required result. (See *The Engineer* for Aug. 24, 1877, p. 139.)

A fall of 2 feet 2 inches on supports 18 feet apart, gives a curve of $\frac{3}{16}$ of an inch, corresponding to a curve of 2,970 feet radius; similarly,

A fall of 2 feet 8 inches gives a middle ordinate of $\frac{3}{8}$ of an inch to the chord of 18 feet, corresponding to a radius of 1,980 feet.

And a fall of 3 feet gives a middle ordinate of $\frac{9}{16}$ of an inch, corresponding to a radius of 990 feet.

For sharper curves a rail-bending machine must be employed. It may often be of service to recollect that the

middle ordinate is equal to the square of the chord, divided by 8 times the radius; the quarter ordinate is equal to $\frac{3}{4}$ the middle ordinate, and the eighth ordinates are equal to $\frac{15}{16}$ and $\frac{7}{16}$ the middle ordinate. Another expression for the middle ordinate is $.22\ n^2\ D$, where D is the degree of the curve and n is the number of 100 feet stations.

In giving the elevation to the outer rail, it, of course, has to be given gradually, and it is the usual custom to begin back of the point of curve a sufficient distance to procure any permissible maximum grade to the rails which lead to the outer side of the curve. (Mr. Froude, quoted by Rankine, says this grade should not be more than one in three hundred. Mr. Geo. W. Kittredge, in *Engineering News* for June 18, 1881, says that one inch in sixty feet, or one in seven hundred and twenty, is found by experience on the Pittsburgh, Cincinnati & St. Louis Railroad, to be as great a grade as is desirable.) If, however, the curve is shifted inwards a certain amount and a new curve begins at some point on it, tangent to it, with a common radius of curvature at that point, but with radii continually becoming longer until it reaches the tangent with an infinite radius at the old point of curve; the elevation can be suited at each point to the curvature at the same point, and the condition that the elevation is always inversely proportional to the radius of curvature, is satisfied. Such a curve is called a "curve of adjustment," and its proper form is stated by Rankine to be an "elastic curve," which possesses the property of starting at a point tangent to a line, and having its radius of curvature at any other point inversely proportional to its distance from the tangent point. The "hydrostatic arch" is such an elastic curve, and could be approximated by a three-centre curve similar to the approximate geostatic arch on a previous page. It is simpler and better, however, to take another curve, which likewise approximates to the elastic curve. Prof. Ran-

kine, in his Civil Engineering, Art. 434, gives the equation of this curve, which may be put in this form :

$$y = \frac{x^3}{6\,e\,R\,k}$$

(this differs from a cubical parabola, because x is measured on the curve, while in the cubical parabola it is a rectilinear co-ordinate) in which x is the distance measured on the circular arc, from the new point of compound curvature to a point whose ordinate from the original shifted curve is y; R is the radius of the circular curve, e is its proper elevation, and k is the reciprocal of the grade which it is determined to give the outer rail to obtain the elevation, recommended above as 300. The distance of the point of compound curvature from the original point of curve is $\frac{k\ e}{2}$ and the amount that the circular curve is to be shifted

over is $\frac{k^2\ e^2}{6\ R}$. If k is taken at 300 and $R\,e = \frac{5730}{24}$, the equation of the curve becomes

$$y = \frac{x^3}{429750}$$

the amount for the curve to be shifted over becomes

$$\frac{855023437\frac{1}{2}}{R^3}$$

and the length of the curve of adjustment becomes

$$\frac{35812\frac{1}{2}}{R}$$

(If $k = 720$, as recommended by Mr. Kittredge, the equation becomes

$$y = \frac{x^3}{1031400}$$

the amount for the curve to be shifted over becomes

$$\frac{4924935000}{R^3}$$

and the length of the curve of adjustment becomes

$$\left.\frac{85950}{R}\right)$$

To apply these equations, when the line is being staked out for the track, the tangents are run in on the original located line; a distance is measured along the original curve from the point of curve and point of tangent equal to the length of the curve of adjustment given above, and stakes are offset from these points on the inside of the curve to the distance the curve is to be shifted over, also given above; between these offset stakes the original curve is to be run in. Between the offset stakes and the point of curve and point of tangent, a curve is to be run in by calculating as many offsets from the circular curve as may be deemed necessary, by means of the equation of the curve of adjustment. One such intermediate stake will usually be sufficient for even very sharp curves. For a curve of 3 degrees, it is seen that the distance the curve is to be moved over is, by the equation, only about one and a half inch; for curves less sharp than this, we need scarcely go to the trouble of putting in a curve of adjustment, but use the original located line, and run out the elevation of the outer rail on the tangent. (If, however, Mr. Kittredge's value of k be used, a 3-degree curve should be shifted about 8½ inches.) At points of compound curvature, a curve of adjustment should likewise be put in, if there is a very great difference in the radii, its length on

each side of the point where the curves approach most nearly, which is approximately the old point of compound curvature, being equal to

$$\sqrt{\frac{6\,a}{\frac{1}{R}-\frac{1}{R'}}}$$

a being the least distance the original curves are apart after shifting, and R and R' are the radii of the curves. It is not often, however, that a sufficient difference exists to make it worth while to calculate the offsets, in the neighborhood of the point of compound curvature, from the shifted curves.

Some engineers, especially in Europe, widen the gauge on sharp curves, in order to lessen the resistance to the motion of the cars. This resistance will be directly as the rigid wheel base, and inversely as the radius of the curve. The additional width of gauge should be proportional to this resistance. A form of equation may then be written for the additional width,

$$w = \frac{a}{R} - b$$

in which a is a constant depending on the rigid wheel base, R is the radius of the curve, and b is a constant depending on the play of the wheels in the gauge, which seems to amount to about $\frac{4}{10}$ of an inch. For a we can write 400, and we then have

$$w = \frac{400}{R} - .4,$$

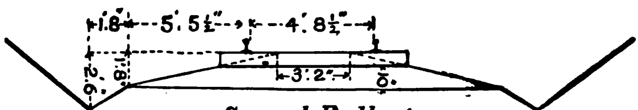
R being in feet and w in inches. This may also be put in the form

$$w = \tfrac{1}{14}\,(\text{degree of curve} - 5.7).$$

It should be observed that this additional width should not in any event be more than one and one-half inch with

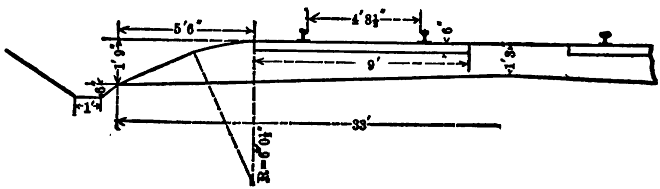
wheels of the ordinary tread (3⅝ inches). This width would correspond by the formula to a curve of 211 feet radius. The rigid wheel base of European cars is 12 to 15 feet, and that of American from 5 to 9 feet. That of the largest locomotives is about 16 feet in Europe and about 14½ in the United States. On the Bavarian railroads the practice seems to be to widen according to a rule which would give similar results to the above formula when 1,115 is substituted for 400 and .1 for .4. In Prussia, the gauge is only widened on curves of less than 1,000 feet radius, and never more than 1 inch in the 4 feet 8½ inches gauge. (Molesworth's Pocket Book.) It will be observed that no widening is required by the formula when the radius is 1,000 feet or more.

The following is the cross-section of the Richmond, Fredericksburg & Potomac Railroad, designed by Mr. E. T. D. Myers, which is recommended for its excellence :

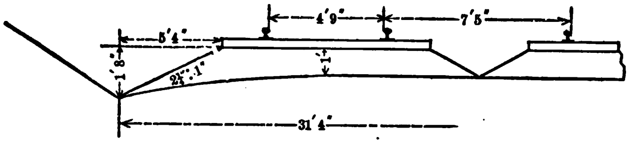


Gravel Ballast.

For comparison we add that of the Michigan Central Railroad, in which the ballast is filled up to the top of the tie, even when of gravel. Most engineers object to this in a country subject to frost, as being apt to lose its line and surface by water freezing in it.



Below is likewise given the cross-section and specifications for the Pennsylvania Railroad, as prepared by Mr. W. H. Brown:



This is for gravel ballast; broken stone ballast is piled up level with the top of the tie, both between the tracks and at the ends.

"SPECIFICATIONS FOR A PERFECT SUB-DIVISION—PENNSYLVANIA RAILROAD.

Superstructure.

"1. The track must be in good surface; on straight lines the rails must be on the same level, and on curves the proper elevation must be given to the outer rail, and carried uniformly around the curve. This elevation should be commenced from 100 to 150 feet back of the point of curvature, depending on the sharpness of the curve, and increased uniformly to the latter point, where the full elevation is attained. The same method should be adopted on leaving the curve.

"2. The track must be in good line.

"3. The splices must be properly put on, with the full number of bolts, nuts, stop-washers and stop-chairs. The nuts must be screwed up tight.

"4. The joints of rails must be exactly midway between the joint ties, and the joint on one line of rails must be opposite the centre of the rail on the other line of the same track. In winter a distance of five-sixteenths of an inch, and in summer one-sixteenth of an inch, must be left between the ends of the rails, to allow for expansion.

" 5. The rails must be spiked both on the inside and outside on each tie, on straight lines as well as on curves.

" 6. The cross-ties must be properly and evenly spaced, sixteen ties to a 30-feet rail, with 10 inches between the edges of bearing surfaces at joints, with intermediate ties evenly spaced a distance of not over 2 feet from centre to centre, and the ends on the outside, on double track, and on the right-hand side going north or west on single track, must be lined up parallel with the rails.

" 7. The ties must not, under any circumstances, be notched, but should they be twisted, must be made true with the adze, and the rails must have an even bearing over the surface of the ties.

" 8. The switches and frogs must be kept well lined up, and in good order. Switches must work easily, and safety blocks must be attached to every switch head.

" 9. The switch signals must be kept bright and in good order.

Road-bed and Ballast.

" 10. The ballast must be broken evenly and not larger than a cube that will pass through a 2½ inch ring. There must be a uniform depth of at least 12 inches of clean broken stone under the ties. The ballast must be filled up evenly between, but not above the top of the ties, and also between the main tracks and sidings, where there are any. In filling up between the tracks, coarse, large stones must be placed on the bottom, in order to provide for drainage, but care should be taken to keep the coarse stone away from the ends of the ties. At the outer end of the ties the ballast must be sloped off evenly to the sub-grade.

" 11. The road crossing planks must be securely spiked, the planking should be three-quarters of an inch below the top of the rail, and 2½ inches from the gauge line. The ends and inside edges of planks should be beveled off.

Ditches.

"12. The cross-section of ditches at the highest point must be of the width and depth as shown on the standard drawing, and graded parallel with the track, so as to pass water freely during heavy rains and thoroughly drain the road-bed.

"13. The lines must be made parallel with the rails, and well and neatly defined.

"14. The necessary cross drains must be put in at proper intervals.

"15. Earth taken from ditches or elsewhere must be dumped over the banks and not left at or near the ends of the ties, but distributed over the slopes. Earth taken out of the ditches must not be thrown on the slope [of a cut].

"16. The channels of streams for a considerable distance above the road should be examined, and brush, drift and other obstructions removed. Ditches, culverts, and box drains should be cleared of all obstructions, and the outlets and inlets of the same kept open to allow a free flow of water at all times.

Policing.

"17. The telegraph poles must be kept in proper position, and trees near the telegraph line must be kept trimmed to prevent the branches touching the wires during high winds.

"18. All old material, such as old ties, old rails, chairs, car material, etc., must be gathered up at least once a week and neatly piled at proper points.

"19. Briers and undergrowth on the right of way must be kept cut close to the ground.

"20. Station platforms and the grounds about stations must be kept clean and in good order."

Many engineers prefer placing the joints opposite each other, and if the track is kept well surfaced, it is probably better. It has the following advantage: The track should be of equal elasticity throughout. The joints are rarely of the same stiffness as the solid rail; by placing the joint-ties nearer together, however, the stiffness of the track at the joints, when they are opposite, may be the same as at the other ties, whereas if the joints alternate, placing the ties nearer together at a joint makes the line of rails at the point opposite more rigid than at the adjacent parts. These remarks apply only to suspended joints. These were adopted because it was found that the rail-heads were less battered at the ends with them than when the joints rested on a tie. It is now said, however, that steel rails do not become battered like iron ones when they rest on a tie. If this is true, and suspended joints are abandoned, it seems that the method of breaking joints with the rails is the true one.

For information in regard to organizing gangs of track-layers in unsettled countries, with a description of the boarding cars for the men, etc., see *Engineering News*, Jan. 21st, April 22d, 1882, Aug. 25th, 1883, and March 15th, 1884.

SWITCHES.

It is usual to designate frogs by numbers which express the relation between the base and altitude of the triangle forming the point of frog. (See Fig. 1.)

Thus a number 8 frog is one whose length is 8 times the base, $DC = 8\,AB$. The numbers in common use are 6, 8 and 10.

Let n = the number of the frog.

" g = the gauge of the track.

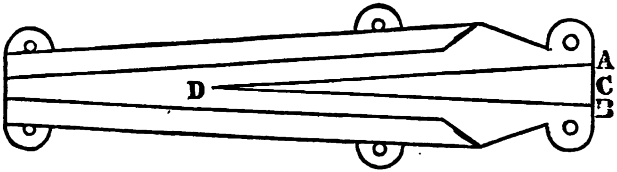


FIG. 1.

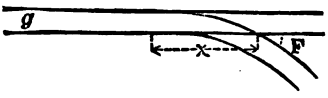


FIG. 2.

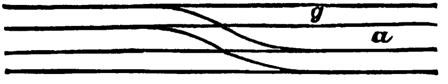


FIG. 3.

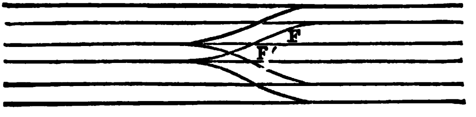


FIG. 4.

To find the distance from the point of curve of the switch rail to the point of the frog on a tangent.

$x = 2\,g\,n.$ (See Fig. 2.)

Radius of turnout $= 2\,g\,n^2$.

When old-fashioned stub switches are used, in order to have a throw of $5\frac{1}{2}$ inches, the movable part of the switch rail should be $1.35\,n\,\sqrt{g}$ long (1.35 being twice the square root of the throw in feet). Five feet are usually spiked fast. A guard rail is placed opposite the frog, 11 feet long and 2 inches in the clear from the traffic rail on the inside, with the ends bent a little towards the middle of the track so as not to catch the wheel.

If the switch is on a curve, the distance from the fixed point of curve of the switch to the point of the frog is:

$$x = R\,g\sqrt{\frac{4\,n^2 + 1}{R^2 + g^2\,n^2}}$$

where R is the radius of the curve; it will not differ much in practice from the value as given on the straight line $x = 2\,g\,n$.

The value of the radius of the turnout is:

$$R' = g\,n^2\left(\frac{2\,R \pm g}{R \mp 2\,g\,n^2}\right)$$

the upper signs being used when the curvature of the turnout is in a different direction from that of the main track, and likewise when the curvature is in the same direction if the turnout runs to the outside of the main track; the lower signs being used when the turnout is to the inside of the main track.

To find the distance from frog to frog in a crossing, the distance measured *along the straight track* from point of frog to point of frog is:

$$e = (a - g)\,n,$$

a being the distance between the tracks. (See Fig. 3.)

To find the proper number for the middle frog in a three-throw switch. (See Fig. 4.)

$$n' = \frac{2\,n^2}{\sqrt{8\,n^2+1}} = .7\,n \text{ approximately.}$$

When two number 10 frogs are used, we find the third frog should be a number 7. When two number 8 frogs are used, the third should be a number 5.6, or say 6. The distance from the point of curve of the switch rail to the first frog in a three-throw switch is

$$x' = 2\,g\,n' = 1.4\,g\,n.$$

Bill of crossing lumber, single switch :

8 ties	9½	feet	long.
2 "	10	"	"
2 "	10½	"	"
2 "	11	"	"
2 "	11½	"	"
1 "	12	"	"
5 "	14	"	"
2 "	15	"	"
1 "	16	"	"

1 switch tie 15 feet long, 16 inches wide.

Bill of crossing lumber, double connection :

6 ties	9½	feet	long.
2 "	10	"	"
4 "	10½	"	"
4 "	11	"	'
4 "	11½	"	"
2 "	12	"	"
16 "	19	"	"

2 switch ties 15 feet long, 16 inches wide.

Bill of crossing lumber, triple connection :

2 ties	9	feet	long.
3 "	10	"	'
2 "	11	"	"
2 "	12	"	"
3 "	13	"	"
1 "	14	feet	long.

2 ties	15	feet long.
1 "	16	" "
3 "	17	" "
3 "	18	" "
4 "	19	" "
2 "	20	" "

1 switch tie 15 feet long, 16 inches wide.

On a double track road, where the modern split switch is used, cross-over tracks should run in the opposite direction to that of the main line, so that trains will have to back in going from one track to the other. This is to avoid what the English call "facing points," which are more unsafe. Where the old stub switch is used, however, it is better to make the cross-over run in the same direction as the train moves.

CROSS TIES.

These should be hewed (not sawed nor split) on two sides, cut square at the ends, and stripped of the bark before delivery. They should be 8½ feet long and 8 inches thick. Three-fourths of them should measure not less than 8 inches across the hewed surfaces, and one-fourth not less than 10 inches. They should be piled in square piles of about 50 each, the ties crossing each other at right angles in alternate layers. Each pile should be separated from the rest, so that a man can pass around each one to inspect the ties. Where one side of the tie is nearer to the heart of the tree than the other, lay it in the track with this side up. In driving the spikes, which are ½ inch square and 6 inches long, the two outer ones are placed 3 inches ahead of the inner ones.

Public road crossings at grade :

The space between the tracks is covered with plank 3⅝ × 8 inches, 16 feet long, spiked to the ties, and leaving 4 inches clear by the rail for the wheel flanges. Planks are also spiked to the ties on the outside of each rail.

RAILS.

A width of 4 inches is sufficient to prevent the rails from cutting into oak ties, and $4\frac{1}{2}$ inches for chestnut ties, when not spaced more than $2\frac{1}{2}$ feet apart. If the base is made more than this, the difficulty of bending the rails to a curve becomes an objection. The stem of an iron rail need not be more than $\frac{1}{2}$ an inch thick, nor that of a steel rail more than $\frac{7}{16}$ of an inch. 60-pound rails are made $4\frac{1}{2}$ and $4\frac{1}{4}$ inches high, 50-pounds 4 inches, and those under 50-pounds $3\frac{1}{2}$ inches.

For further information and sections of rails, reference may be made to the Reports in the Transactions of the American Society of Civil Engineers, Vol. 3, p. 106, and Vol. 4, p. 136, and Vol. 21, pp. 133, 153 and 223, as well as to the *Railroad Gazette* for November 27th, 1875, and March 11th, 1881. The committee of the American Society of Civil Engineers recommends that the radius of the top of the rail should be 12 inches for rails of all weights, with a wide head, its mass approaching equality with that of the flange, with the corner rounded with a radius of $\frac{1}{4}$ inch, vertical sides, and a lower corner radius of $\frac{1}{16}$. (See *Engineering News* for March 21st, 1891, p. 278.) Mr. Welch, in the second of the Reports, gives the following as the rules to be observed in the manufacture of iron rails:

"Select the stock best adapted to each part, the hardest metal for the head, the strongest for the base. Use only gray metal, not white. Put no old rails into the head or base; puddle thoroughly, or the metal will not weld thoroughly. Cut off and throw out all ends of puddle bars; make the top slab about $1\frac{1}{4}$ inch thick; thicker will not heat before the bars burn; pile 8 inches square. The top slab should be four rolled (thrice heated), the bottom thrice rolled, and the stem twice rolled; no puddle bars in

the rail. Each heating should be uniform and thorough, without burning, or the welding will be imperfect."

The following are the specifications for steel rails adopted on the Pennsylvania Railroad by Mr. W. H. Brown :

"As it is the desire of the Pennsylvania Railroad Company to have on the roads under their control none but first-class tracks in every respect, and as the rails laid down on these tracks form an important part in the achievement of this result, the Pennsylvania Railroad Company have found it necessary to make certain demands in regard to the manufacture of their steel rails, with which the different rolling mills and rail inspectors will be required to comply.

"1. That the steel for rails shall be made in accordance with the 'pneumatic' or the open-hearth process, and contain not less than $\frac{30}{100}$ nor more than $\frac{50}{100}$ of 1 per cent. of carbon.

"2. The result of the carbon test of each charge, of which the Pennsylvania Railroad Company is to receive rails, and of which an official record is kept at each mill, is to be exhibited to the rail inspector.

"3. A test bar, three-quarters of an inch wide and about 10 inches long, to be taken from the web of a rail made from each charge, is to be furnished to the railroad company's inspector, for use in making analysis and test of the steel, whenever required.

"4. The number of the charge and place and year of manufacture shall be marked in plain figures and letters on the *side* of the web of each rail.

"5. The weight of rails shall be kept as near to standard weights adopted by the Pennsylvania Railroad as it is practicable to do so.

"6. The sections of the rails shall correspond with the respective templates, showing the shape and dimensions of

the different rails adopted as standard as near as practicable after complying with section 5.

"7. The space between the web of the rail and *template*, representing the splice-bar, shall not be less than $\frac{1}{4}$ of an inch, nor more than $\frac{3}{8}$ of an inch.

"8. Circular holes, 1 inch in diameter, shall be drilled through the web in the centre thereof, at equal distances from the upper surface of the flange and lower surface of the head, and $3\frac{5}{16}$ inches from the end of the rail to the centre of the first hole, and of 5 inches from the centre of the first hole to the centre of the second hole.

"9. The length of rails at 60° Fahrenheit shall be kept within $\frac{1}{4}$ of an inch of the standard lengths, which are 30 feet, $27\frac{1}{2}$ feet, and 25 feet. When specially mentioned in the contract, and not otherwise, 10 per cent. of rails of shorter lengths and 5 per cent. of *second-class* rails will be accepted.

"10. The rough edges produced at the ends of the rails by the saw shall be well trimmed off and filed.

"11. All rails are to be straightened in order to insure a perfectly straight track.

"12. The causes for a *temporary* rejection of the rails are:

"1. Crooked rails.

"2. Imperfect ends (which, after being cut off, would give a perfect rail of one of the standard short lengths).

"3. Missing test reports.

"4. A variation of more than $\frac{1}{4}$ of an inch from the standard lengths.

"13. The causes for a *permanent* rejection of a rail as a first-class rail are :

"1. A bad test report, showing a deficiency or excess of carbon.

"2. The presence of a flaw of $\frac{1}{4}$ of an inch in depth in any part of the rail.

"3. The presence of such other imperfection as may involve a possibility of the rail breaking in the track.

"4. A greater variation between the rail and splice-bars than is allowed in paragraph No. 6."

Dr. Charles B. Dudley, of the Pennsylvania Railroad, found from analyses of the best steel rails that the following are the best proportions of the elements :

Limit of phosphorus, $\frac{1}{10}$ per cent.
" silicon, $\frac{4}{100}$ per cent.
" carbon, $\frac{25}{100}$ to $\frac{35}{100}$ per cent.
" manganese, $\frac{30}{100}$ to $\frac{40}{100}$ per cent.

The average of good rails was : Carbon, .287 ; phosphorus, .077 ; manganese, .369, and silicon, .044. The tensile strength should be above 65,000 pounds per square inch, and the elongation before breaking about 30 per cent., the ductility, as indicated by this last quality, being specially important. These proportions of the elements, however, are not accepted by other engineers as being exact ; there may be more phosphorus if the amount of manganese is also increased. Sulphur and copper have not been stated, because if there is enough of either to do harm the steel will be "hot short," and cannot be rolled. (See discussion in Transactions American Society of Mining Engineers, Vols. VII. and VIII.) If there should be neither phosphorus nor silicon the rail would be too soft, unless the proportion of carbon was raised to about .5.

There seems to be a tendency at present to make rails harder by increasing the amount of carbon above what has been recommended for a good rail, though it is at the expense of ductility. The increased strength required is obtained by increasing the weight of the rails even up to 80 pounds per yard. On a ship railway in Nova Scotia, rails weighing 110 pounds per yard are used. Some suggest that more work be put upon the rail in its manufacture by increasing the number of passes in the rolls, and likewise that it be finished at a lower heat, both of which would increase its strength. The Lehigh Valley Railroad

is experimenting with rails containing .42 to .66 per cent. of carbon. The hardening elements in the order of value, the inverse order of their producing brittleness, are, carbon, manganese, silicon, phosphorus.

In the manufacture of rails by the Bessemer process, it is of great importance that the heat of the converter should be at a particular degree. If too hot, the carbon burns before the silicon and the latter is left in excess, making the rails brittle; if too cold, on the other hand, the iron oxidizes before the manganese, leaving the latter in excess. The temperature is regulated by putting scrap steel in the converter. A leading manufacturing company adopts for its rails the proportions of

Carbon, .30 to .40 ;
Silicon, .10 to .15 ;
Phosphorus, .10 ;
Sulphur, .05 to .10 ;
Manganese, .90 to 1.00 ;

Dr. Dudley likewise recommends that for a mechanical test, pieces be cut from the web of the rail 12 inches long, $1\frac{1}{2}$ inch wide and $\frac{1}{2}$ inch thick, and supported on "knife edges" 10 inches apart, and bent by a knife edge at equal distances from the supports; they should stand not over 3,000 pounds, and bend not less than 130° without rupture.

The tests applied to rails in Germany are as follows: They must sustain 20 tons without permanent flexure, when resting on supports 1 metre apart; they must bear two strokes of 1,100 pounds falling 13 feet without breaking, and one stroke of the same weight falling 5 feet without injury; they must also bend cold 2 inches without cracks, and curve $\frac{9}{10}$ of an inch in 9 feet 10 inches. For other rail tests see *Engineering News*, Nov. 28th, 1885, p. 342.

Steel rails will stand a traffic of about 63,000,000 tons

before wearing out. (Some authorities, however, think they ought to stand three times this amount.)

In the inspection of rails the template of the rail is applied to see that the section is correct. A rail is also weighed as it comes from the straightener, at intervals of about four hours, to see that the rolls are at the proper distance apart. It should not vary more than 1 per cent. from its proper weight as calculated from the section. When the rolls are first put in the rails may weigh 1 per cent. light, but wear on them will make the distance between them greater, and in a large order the rails will in time weigh a little heavier than the normal amount. This is in favor of the buyer, payment being made on the calculated weights, and is not allowed to remain long by the manufacturer, who screws the rolls together again.

The individual rails are inspected as they are being loaded on cars. They are placed upright on skids alongside of the cars until they are full, holding, perhaps, 50 rails. The inspector sights over them from one end to the other, and if one is not straight, he chalks it to go back to the straightener. In doing this he must be especially anxious to detect and throw out those that have two bends in opposite directions, even though very slight, as they cannot be removed by the tracklayers. If the bend is only in one direction, the rail can be spiked straight when laid, but it is better to send all crooked rails back to be straightened. The inspector looks at the same time for flaws in the head of the rail, and is very particular in throwing out a rail in which he finds one. To detect these he will probably have to walk over the tops of the rails, as it is difficult to detect them from the ends. After he has completed this inspection the rails are all turned over on their sides, and he looks at the bottom of each one as it is pushed past him to the car. By standing so that the light is reflected from them he can detect little nicks in

the edges, which when over a quarter of an inch, he condemns; or when a succession of little nicks indicates bad metal. When a rail is thrown out for such a notch in the flange, it goes to the saw, where the nicked portion is sawed off. The remainder is put with the others as a short rail, the contract usually allowing the manufacturer to put in 10 per cent. of short rails.

The inspector also sights along the rails when on their sides, to see that they are not given too much "sweep," as it is sometimes called, that is, are not curved up too much. If the bottom of the rail is convex in the direction of its length, it is sent back to the straightener; if it is slightly concave, it is accepted, some persons accepting those with a considerable upward curve. When laid on level ties, their weight will make them level, but one that has ends curving upward will have to be kept level on the ties by the spikes, and is constantly exerting an upward pull on them, and if the spikes loosen, will prove dangerous; therefore, such rails are sent back to the straightener.

As a check on the manufacturer to see that he gives full sections, the weight of the shipments may be got from the scale office, where the weights of the empty and loaded cars are recorded, from which the freight is calculated by the railroad company that carries them.

WATER STATIONS.

Passenger engines on the Middle Division of the Pennsylvania Railroad, where the grades are very light, run at a rate of 35 miles per hour with seven cars; and, when making frequent stops, one tank of water, containing 2,400 gallons, lasts for two and a quarter hours, or 78 miles. The engines, however, take in water actually every 45 miles. A freight train on the same division, with a full

tank, can run at a speed of $14\frac{1}{2}$ miles an hour for two hours fifty minutes, or $41\frac{1}{2}$ miles, with one tank of water. As, however, they have to stop at shorter intervals to allow passenger trains to pass, or to pass each other, they utilize the time of waiting in filling their tanks. On the Mountain Division of the same railroad, freight trains, with a full load on an almost continuous grade of $1\frac{9}{10}$ per cent., use a tankful of water, containing 3,000 gallons, in one hour fifteen minutes, or in going 15 miles. It is thus seen that 15 miles is the extreme distance apart for water stations with grades of 2 feet in 100, while 40 miles would do with very light grades. It would be well, however, if water is plenty, to have them every 10 miles, or oftener.

In a hilly country, streams can generally be dammed up, which will give a gravity supply. The outer slope of the dam may be built of stone, like a retaining wall, or may be of earth at its natural slope. The inner slope should be at the natural slope of clay in water, which is 3 to 1. There should be a layer of good clay on the inside, 2 feet thick if the reservoir is less than 16 feet deep; if of a greater depth, make the thickness of the puddle equal to $\frac{1}{8}$ the depth of water. Cast-iron pipe, 6 inches in diameter, is used to convey the water from the dam to the track, a smaller diameter being too liable to be stopped up. For calculating the quantity of water delivered, Darcy's formula gives very good results; for rusted pipes

$$Q = 31.6\, d^2 \sqrt{\frac{d\,h}{l}}$$

where Q is the quantity in cubic feet per second, d is the diameter of the pipe in feet, and $\frac{h}{l}$ is the average grade of the pipe. It is better that the pipe should have a continuous down grade. If however, this is impossible, it may be laid undulating so long as no portion rises above

the "hydraulic grade line," which is the straight line drawn from the surface of the water in the dam above the inlet to the exit from the pipe at the side of the track. If it is so laid undulating, it will be necessary to place an air cock at every "summit," and a mud cock (blow-off cock) at every "valley." At the track, a "stand pipe" or "plug" is placed, which rises to an outlet 9 feet above the rail. A valve controls the outlet, within reach of the engine driver. A piece of rubber hose, 7 inches in diameter, 10 feet long, is fastened on the end of the pipe, to insert into the opening in the tender.

It may often happen that a dam cannot be made, or there is not enough water in the stream to furnish a continuous supply. A tank is then placed at the side of the railroad. This is a tub made of white pine, 18 feet in diameter at the bottom and 17 at the top, 8 feet deep. The bottom is 3 inches thick and the staves $2\frac{1}{2}$ inches thick. There are 6 iron hoops, $\frac{1}{4} \times 3$ inches; two placed close together at the base, and the others at intervals increasing toward the top. The bottom is let into a groove in the staves, but the ends of the staves are let into the floor, so that the bottom bears over its whole surface on the floor. The tub is supported on three trestles of 10×10 inches stuff, placed 6 feet 6 inches apart, on walls 18 inches thick, built parallel to the track, and finished off 1 foot above the rail. On these trestles, joists 4×12 inches are placed 1 foot apart, which support the floor of 2-inch stuff, on which the bottom of the tub directly rests. Where a greater supply is required, or a more permanent structure, and an adjacent hill permits it, stone reservoirs are made, 40 feet in diameter and 8 feet deep. They are built below the surface of the ground. The walls are built of common mortar, with a lining of brick, well wet and thoroughly bedded in cement. The bottom is covered with a layer of stones about the size of a walnut, 4 inches deep, and made into a

concrete with cement; and when it is set another layer of the same thickness is put in. These reservoirs are covered with a house.

Iron tanks are often used at the side of the track, instead of the wooden tub. They may be made 7 feet in diameter and 7½ feet high, made up of three rings, with the bottom and lowest ring of ¼-inch plate iron, the middle ring of $\frac{3}{16}$ and the top of ⅛-inch iron.

Where a gravity supply cannot be obtained, water must be pumped into the reservoir with a steam engine or windmill.

Six-inch pipe, of a thickness of $\frac{7}{16}$ of an inch, is made to lie in lengths of 12 feet. Each joint requires 8 pounds of lead and ¼ of a pound of a "gasket" of loosely twisted rope, which comes for the purpose.

COALING STATIONS.

A freight engine with its load consumes about 160 bushels of bituminous coal in going 131 miles, and tenders hold about 80 bushels, or 8,000 pounds.

This will give some basis for calculating the distances at which coaling stations must be provided. This, however, is a larger amount than the best engines ought to require. In England, on heavy grades and sharp curves, passenger engines consume about 49 pounds of coal per train mile, and on level roads about 27 pounds; while freight engines consume about 43 pounds with a load of 438 tons at 20 miles per hour over heavy grades and sharp curves. The weight of the coal burnt will be from one-fifth to one-seventh of the weight of the water evaporated. In 1884 the average amount of coal burned on the Pennsylvania Railroad was 99.2 pounds per train mile of freight trains, the average load being 271 tons of freight and 295 tons of cars, or a total of 566 tons. Passenger engines burned

53.6 pounds per mile per train. (*Railroad Gazette,* Sept. 11th, 1885.) Some observations reported in the *Railroad Gazette* for May 7th, 1886, give 21 pounds of coal per mile for the engine and nearly 7 more for each car in train.

On the 2 per cent. grades of the Pennsylvania Railroad, however, 9 pounds of coal per car per mile are required with 6 cars, while on the lighter grades of ½ per cent. only 2¾ pounds are required with 50 cars.

On 1 per cent. grades an engine will pull a train of 9 times its own weight, with that of its tender, in all weathers. On 2 per cent. grades, with the same loads, although it will haul them, the steam pressure will be reduced with ordinary engines. It will, on the latter grade, haul about 5 times its own weight continuously over sharp curves.

PASSENGER STATIONS.

Description of Cresson Passenger Station, Pennsylvania Railroad:

One story high, 70 × 40 × 12 feet high, with sloping roof. Posts or "studs" are set 6 × 7 inches at the corners, and 5 × 6 inches at points 5½ feet apart. These are braced by horizontal pieces, 3 × 4 inches, placed about 4 feet apart, except at the windows and doors. Diagonal braces, 4 × 6 inches, are placed at the upper corners, framed into the posts and a beam 6 × 7 inches, which forms the tie-beam of the roof truss. The latter is a king post of 6 × 6 inch pieces, with secondary king-post trusses abutting toward the centre against a straining beam, all 4 × 6 inches. The ridge pole is 2 × 10 inches, with purlins 4 × 7, and rafters 3 × 5 inches, spaced 2 feet apart. Joists, 3 × 10, spaced 1½ feet apart. Flooring, 1¼ inches, worked. Roof sheeting, 1 inch. Platform flooring, 2 inches. Platform joists, 3 × 9½ inches. Weather board-

ing, $\frac{7}{8}$ of an inch thick, 9 inches wide, with $\frac{7}{8}$ of an inch stripping, 2 inches wide. Partition of $\frac{7}{8}$ of an inch stuff. Plastering lath, 3 feet long.

Water station at Gallitzin, Pennsylvania Railroad:

"Balloon frame," 22 feet 8 inches by 22 feet 8 inches by 18 feet 6 inches high. Wall plates, 3 × 8 inches. End posts, 4 × 4 inches. Studs, 2 × 4 inches, 18 inches apart. Diagonal pieces, $1\frac{1}{2}$ × 3 inches, 2 feet 6 inches apart, measured vertically. Rafters, 2 × 6 inches. Ridge pole, 2 × 8 inches. Joists, 4 × 12 inches. Siding of $\frac{7}{8}$ of an inch worked boards, tongued and grooved. Sheeting, ditto. Slate roof.

It may here be remarked that when a plank is nailed to a post or joist, or other wooden substance, a nail is used of such a length that it will go twice as far into the post or joist as the thickness of the plank. Thus, for $\frac{7}{8}$-inch stuff use $2\frac{1}{2}$ inches long or 8-penny nails, for 1-inch stuff use 3 inches long or 10-penny nails.

TELEGRAPH LINE.

The number of poles to the mile varies from 26 to 42. The size of wire varies from 320 to 380 pounds per mile. The poles should not be less than 5 inches in diameter at the top, nor less than 25 feet long. If green, they should be charred 5 feet from the bottom. If any are split at the lower end, the parts should be nailed together before putting in the ground; otherwise, the spring of the wood will prevent the earth from packing around them. The cross-arms are made of 3 × 4 inches, 3 feet long, white pine, painted white, one bolt for each cross-arm, $\frac{1}{2}$ inch diameter, 8 inches long, square head and nuts, and wrought washers.

Sixty miles of line will require at each end a battery of 15 cups (Grove's). These cups require to be re-charged

twice a week. A battery of 30 cups requires one carboy or 200 pounds of nitric acid, 25 pounds of sulphuric acid, and 1 pound of zinc per cup, every month.

APPENDIX.

The following are the specifications from which the Columbia & Port Deposit Railroad was built:

GRADUATION.

1. Under this head will be included all excavations and embankments required for the formation of the road-bed; cutting all ditches or drains about or contiguous to the road; the foundations of culverts, and all small bridges or walls; the excavations and embankments necessary for reconstructing turnpikes or common roads, in cases where they are destroyed or interfered with in the formation of the railroad; and all other excavations or embankments connected with or incident to the construction of said railroad.

2. All cuttings shall be measured in the excavations and estimated by the cubic yard under the following heads, viz.: Earth, loose rock, solid rock.

Earth will include clay, sand, loam, gravel, and all other earthy matter, or earth containing loose stone or boulders intermixed which do not exceed in size 3 cubic feet.

Loose rock shall include all stone and detached rock, lying in separate and contiguous masses, containing not over 3 cubic yards; also all slate or other rock that can be quarried without blasting, although blasting may be occasionally resorted to.

Solid rock includes all rock occurring in masses exceeding 3 cubic yards, which cannot be removed without blasting.

3. The road will be graded for a single track, except where otherwise directed by the Engineer, with a road-bed of such width, and side slopes of such inclination as the Engineer shall in each case designate, and in conformity to such depths of cuttings and fillings as may have been or may hereafter be determined upon by said Engineer.

4. Earth, gravel and other materials taken from excavations (except when otherwise directed by the Engineer), shall be deposited in the adjacent embankment, the cost of removing which, when the haul is not more than 1,400 feet, will be included in the price paid for excavation ; all material necessarily procured from without the road and deposited in the embankments will be paid for as embankment only, but all material necessarily procured from within the line of the railroad, and hauled more than 1,400 feet, will be paid for as excavation and also as an embankment. In procuring materials for embankment from without the line of the road, the place will be designated by the Engineer in charge of the work, and in excavating and removing it, care must be taken to injure or disfigure the land as little as possible. The embankment will be formed in layers of such depth, and material disposed and distributed in such manner as the Engineer may direct, with the required allowance for settling.

Materials necessarily wasted from the cuttings will be deposited in the vicinity of the road, according to the directions of the Engineer in charge, and if, during the progress of the work, the raft channel of the Susquehanna River should be obstructed, by blasting of rocks, sliding of earth, or from any other cause, the same shall be carefully removed, without delay by the contractor, at his own expense.

5. The ground to be occupied by the excavations and embankments, together with a space of 12 feet beyond the

slope stakes on each side, or 10 feet beyond the berm ditch, where one is required, will be cleared of all trees, brush and other perishable matter. Where the filling does not exceed 3 feet, the trees, stumps and saplings must be grubbed, but under all other portions of the embankment it will be sufficient that they be cut close to the earth. No separate allowance will be made for grubbing and clearing, but its cost will be included in the price for excavation and embankment.

6. Contractors, when directed by the Engineer in charge of the work, will deposit on the side of the road, or at such convenient points as may be designated, any stone or rock that they may excavate, and if in so doing they should deposit material required for embankment, the additional cost, if any, of procuring other materials from without the road will be allowed. All stone or rock excavated and deposited as above, together with all timber removed from the line of the road, will be considered the property of the Railroad Company, and the contractors upon the respective sections will be responsible for its safe keeping until removed by said Company, or until the work is finished.

7. The line of road or the gradients may be changed, if the Engineer shall consider such change necessary or expedient, and for any considerable alterations, the injury or advantage to the contractor will be estimated, and such allowance or deduction made in the prices as the Engineer may deem just and eqnitable; but no claim for an increase in prices of excavation or embankment on the part of the contractor will be allowed or considered, unless made in writing, before the work on that part of the section where the alteration has been made shall have been commenced. The Engineer may also, on the conditions last recited, increase or diminish the length of any section for the purpose

of more nearly equalizing or balancing the excavations and embankments.

8. Whenever the route of the railroad is traversed by public or private roads, commodious passing places must be kept open and in safe condition for use; and in passing through farms the contractor must also keep up such temporary fences as will be necessary for the preservation of the crops.

MASONRY.

All masonry will be estimated and paid for by the perch of 25 cubic feet, and will be included under the following heads, viz.: Rectangular and arch culvert masonry, first and second quality bridge masonry, vertical and slope wall masonry and paving.

1. *Culvert Masonry.*—All rectangular culverts will be built dry (not less than $2\frac{1}{2} \times 3$ feet) as may be required by the Engineer; the abutment will rest on a pavement of stone, set edgewise, of at least 10 inches in depth, confined and secured at the ends by deep curb-stones, which will be protected from undermining by broken stone, placed in such quantity and position as the Engineer may direct. The abutment walls will be not less than 2 feet thick, and built of good-sized, well-shaped stone, properly laid and bound together by stones, occasionally extending entirely through the walls. The upper course to have at least one-half of the stones headers; and the stretchers in no case to be less than 12 inches wide; no stone in this course to be less than 6 inches thick. The covering to be of sound, strong stone, at least 12 inches thick, and to lap not less than 10 inches on each abutment. The thickness of the covering stone and dimensions of the walls to be increased at the discretion of the Engineer, according to the height of the embankment and span of the culvert.

2. *Semi-Circular or other Culverts with Curved Arches.*—The foundations of these culverts, when the bottom of the pit is common earth, gravel, etc., will generally consist of a pavement formed of stone set edgewise, not less than 12 inches in depth, secured in the same manner as before described for rectangular culverts. When the foundation upon which a culvert is to be built is soft and compressible, and where it will at all times be covered with water, timber well hewed and from 8 to 12 inches in thickness (according to the span of the culvert) will be laid side by side crosswise upon longitudinal sills, and where a strong current will be forced through during floods, three courses of sheet piling are to be placed across the foundation, one course at each end and one in the middle, to be sunk from 3 to 6 feet below the top of the timber, according as the earth is more or less compact. The abutments are to be built of rubble work, the stones hammered on their beds and laid in courses; the stretchers in the face are to have beds of at least 15 inches, and they are to be not less than 2 feet long, measuring in the face of the wall; the headers will extend through the wall in cases where it does not exceed 3½ feet thick, and they shall have not less than 18 inches length of face. There shall be not less than one header to every 7 feet of face, measured from centre to centre, and so arranged that a header in a superior course shall be placed between two headers in the course below; the backing stone shall be of large size and have parallel beds, laid so as to break joints with one another, and when the thickness of the wall exceeds three and a half feet, headers of the same dimensions as those in the face will be placed in the back of the wall in the proportion of one for every two headers in the face. The beds and joints of the arch stone are to be dressed so as to give an even bearing on each other, and to be laid in

courses throughout. The ring stone will be neatly cut and composed of alternate long and short bond stones of not less than three feet, and eighteen inches respectively. The parapet and wing walls will be built similarly to the abutments, and surmounted with a well-dressed coping not less than ten inches thick and three feet wide.

3. *Bridge Masonry.*—When rock foundation cannot be had for abutments and piers, the masonry shall be started upon hewn timber, sunken to such a depth as to protect it from decay and to prevent the possibility of underwashing. The timber platforms will be composed of one or more courses, according to the depth of the water, the height of the masonry, or other circumstances of which the Engineer shall judge and determine. The masonry will be of two qualities, either to be adopted at the *discretion* of the Engineer. First quality will be rock range work. The stone to be accurately squared, jointed and bedded, and laid in courses of not less than twelve inches thick, nor exceeding twenty inches in thickness, regularly decreasing from bottom to top of pier or abutment. The stretchers shall in no case have less than sixteen inches bed for a twelve-inch course, and for all courses above sixteen inches, at least as much bed as face; they shall generally be at least four feet in length. The headers will be of similar size with the stretchers, and shall hold the size in the heart of the wall, that they show on the face, and be so arranged as to occupy one-fifth of the face of the wall, and they will be similarly disposed in the back. When the thickness of the wall will admit of their interlocking they will be disposed in that manner. When the wall is too thick to admit of that arrangement, stones not less than four feet in length will be placed transversly in the heart of the wall to connect the two opposite sides of it. The stones for the heart of the wall will be of the

same thickness as those in the face and back, and must be well fitted to their places; any remaining interstices will be filled with small stones or chips. The face stones will, with the exception of the draught, be generally left with the face as they come from the quarry, unless the projections above the draught should exceed two inches, in which case they shall be roughly scabbled down to that point. The abutments or piers, and such portions of them as the Engineer may direct, shall be covered with a course of coping not less than twelve inches thick, well dressed, and fastened together with clamps of iron.

The second quality of bridge masonry will be rubble work, and will consist of stones containing generally six cubic feet each, so disposed as to make a firm and compact work, and no stone in the work shall contain less than two cubic feet except for filling up the interstices between the large blocks in the heart of the wall; at least one-fifth of the face shall be composed of headers extending full size four feet into the wall, and from the back the same proportion and of the same dimensions, so arranged that a header in the back shall be between two headers in the face. The corner stones shall be neatly hammer-dressed, so as to have horizontal beds and vertical joints.

4. *Vertical and Slope Wall.*—The vertical walls will be good dry rubble work, of such dimensions, and built with such batter, as the Engineer may direct. Slope walls will be built of such thickness and slope as may be required by the Engineer. No stones, however, to be used in its construction which do not reach through the wall, nor any less than six inches in thickness by twelve inches long; the bed of the stone to be placed at right angles with the face of the bank, the joints close and free from spalls.

5. In all masonry the stones must be of a hard and durable quality, of good size and shape to be approved of by

the Division or Principal Assistant Engineer. Such portions of the masonry as the Engineer may require to be laid in lime mortar or hydraulic cement will be so laid, the Railroad Company furnishing or paying for the lime and cement used. If in the progress of the masonry, an increase in the number of headers specified should be required by the Engineer, such additional number shall be laid in the work as he shall designate.

6. The price per perch for masonry shall in every case include the furnishing of all materials (except lime and cement); the cost of scaffolding, centering, etc., and all expenses attending the delivering of these materials and all risks from floods or otherwise.

7. *Rip Rap.*—All rock in excavation to be deposited upon the river side of embankments, within the distance considered an overhaul, of such depth, and at such places as the Engineer may direct, without any extra compensation being allowed, excepting whenever the party of the first part may, or shall be required by the party of the second part, or the said Engineer, to use the material out of the rock cuts, for rip rapping any embankments that may have been made of earth, he shall be allowed a price per cubic yard for such rip rap, in addition to the rock price. If an overhaul, it shall be paid for as excavation and embankment. In classification, it shall be known as "rip rap from excavation."

Where the excavation of the road bed does not furnish sufficient stone for the protection of walls and embankments, the same shall be procured, at such places and disposed in such manner as the Engineer may direct, and a price per cubic yard paid therefor.

8. *Ballast.*—The ballast must be of good hard stone, to be approved by the Engineer. It must be well broken into cubical pieces of such size as to pass through a ring of three inches in diameter. It must be placed on the road

bed, of such width and depth as the Engineer may direct.

9. The quantities exhibited to the contractor at the letting are, from the necessity of the case, *merely approximate;* they furnish only general information, and will in no way govern or affect the final estimate of the work, which will be made out on its completion, from actual measurements and established facts, not now in the possession of any one, nor possible to be obtained at the time of drawing up this specification.

10. No charge shall be made by the contractor for hindrances or delay, from any cause, in the progress of any portion of the work in this contract, but it may entitle him to an extension of time allowed for completing this work sufficient to compensate for the detention, to be determined by the Chief Engineer, provided he shall give the Engineer in charge immediate notice in writing of the cause of the detention.

Nor shall any claim be allowed for extra work unless the same shall be done in pursuance of a written order from the Engineer in charge, and the claim made at the first settlement after the work was executed, unless the Chief Engineer at his discretion should direct the claim or such part as he may deem just and equitable to be allowed.

11. Any work which the Engineer in charge may require done, under this contract, and for which there is no specific price named herein, shall be paid for on the estimate, and at a value fixed by said Engineer, subject to the approval of the Chief Engineer.

For other specifications see *Engineering News*, Aug. 25th, 1883.

NOTES ON A CONTRACTOR'S WORK.

According to Rankine, the proper number of wheelers to one shoveller is

$$\frac{l + 6\,h}{100 \text{ to } 120}$$

in which l = the horizontal distance that the earth has to be wheeled, and h = the height of ascent, if any, that the earth has to be wheeled.

The number of pickmen to one shoveller for loose sand and vegetable mould = 0, for compact earth = $\frac{1}{2}$, for ordinary clay = $\frac{1}{2}$ to 1, and for hard clay = $1\frac{1}{2}$ to 2.

According to Mr. Ellwood Morris, the cost of a cubic yard of embankment in cents is:

$$e + \frac{c}{g} + \frac{d\,f\left(\frac{a}{100} + 4\right)}{60\,b} + 1$$

in which a = the number of feet in the average haul.

b = the number of hours worked per day.

c = the daily wages in cents.

d = the daily wages of a cart, including driver.

e = the cost of loosening materials in cents per cubic yard, ranging from 1 to 8.

f = the number of cart loads required to form a cubic yard, usually 3 on a descending road, $3\frac{1}{2}$ on a level, and 4 on an ascending road.

g = the cubic yards which a medium laborer will load into a cart per day, ranging from 10 to 14.

Mr. Trautwine makes the 1 of the formula $1\frac{1}{2}$ for clay, and adds 1 for keeping the cart road in good order and 2 for wear, sharpening and depreciation of picks, superintendence, water carriers and trimming, and the formula then reads,

Cost per cubic yard of excavation in cents =

$$e + \frac{c}{g} + \frac{d\,f\left(\frac{a}{100} + 4\right)}{60\ b} + 4\tfrac{1}{3}$$

for clay and with one driver to four carts.

For barrows, cost per cubic yard =

$$e + \frac{(c + 5)\ 14\left(1.25 + \frac{a}{100}\right)}{54\ b} + 4\tfrac{1}{3}.$$

For rock read 1.6 for 1.25.

A two-wheel cart holds as much as 5 wheelbarrows.

The run-way of barrows, if inclined, should be not more than 1 in 12. If the earth is to be wheeled more than 80 feet, divide it into stages, each one 80 feet long, and employ separate wheelers for each stage. When more than four stages are necessary, use horses and carts. The inclination of a road for a horse and cart should not exceed 1 in 20. When earth is thrown up by stages, each one should be five feet high.

In blasting rock, the holes are made from 1 to 3 inches in diameter, and 1 to 4 feet deep. One man can drill from 80 to 160 cubic inches of rock per day in granite, and five times as much in limestone. The drill requires to be sharpened about once for every foot bored, and to be steeled once for each 16 to 20 feet. To produce the same effect with dynamite as with powder requires only about from one-eighth to one-third (in tunnels) the weight. For blasting with common powder the following will indicate the amount required :

$$\text{Charge in ounces} = \frac{\text{cube of line of least resistance in feet.}}{2}$$

The amount is easily found from the following table :

Diameter of hole in inches.	Depth in inches to contain one ounce of powder.
1	2.39
1½	1.06
1¾	.78
2	.60
2½	.38
3	.27

The ordinary rule is to fill one-third of the depth of the hole with powder, and tamp the rest with clay. Having, then, estimated the length of the line of least resistance of a proposed blast, find first the charge from the equation, then the depth of the hole of the proposed diameter to contain the charge by the table, and multiply by three for the depth of hole required. In small blasts one pound of powder will loosen about 4½ tons of rock, and in large blasts about 2¾ tons. According to M. Chalon, if no tamping is used, but the hole is stopped by a piece of clay 6 inches long, fitting the hole tight but leaving an air space over the powder, a much greater rending effect is produced than in the ordinary way with black powder. (See *Engineering News,* Vol. 18, p. 261.)

The holes are usually started with a diameter of 2 or 2½ inches; each time that the drill has to be sharpened its width is measured, and the new width made the same as that of the worn edge. The hole thus becomes of a tapering diameter downwards, the taper becoming faster the harder the rock. The hole will usually take a triangular shape, with curving sides, and the blast will split up the rock by cracks proceeding from the corners. The drill is made chisel-shaped for hard rock, but when the rock is in seams, a cross-shaped edge is generally preferable.

The following are the rules of Schoen for tunnels, reduced to English measures :

Let f = the area of the tunnel in square feet.
" l = the depth of the hole in feet.

Let d = the diameter of the hole in inches.
" L = the charge of powder in ounces.
Then $d = .92 + .24\,l$.

$$L = l\,(1.538 + .787\,l + .1\,l^2).$$
$$l = .2\,\sqrt{f}.$$

If $f = 380$; $l = 3.9$, $d = 1.86$, $L = 24$.
If $f = 81$; $l = 1.8$, $d = 1.35$, $L = 5.9$.

When dynamite is used, the holes are made of two and a half inches diameter and about four to six feet deep, and one or two cartridges are used according to the kind of rock, each cartridge containing half a pound of the explosive. Under water, no tamping is needed. In tunnels the holes are made from seven to ten feet deep. In the Delaware, Lackawanna & Western R. R. tunnel, built in 1883, six 3-inch Ingersoll drills were used, four in the headings and two on the benches. For driving these, one Ingersoll air compressor was used, and three Southers 20 horse power boilers. The progress was 502 feet in a month. (*Engineering News*, Vol. 10, p. 371.)

Approximate prices of work in 1892: Earth excavation, 18 cents per cubic yard; "hard pan" excavation, 30 cents per cubic yard; loose rock, 40 cents per cubic yard; solid rock, 80 cents per cubic yard; concrete, \$3 to \$3.75 per cubic yard; culvert masonry, dry, \$3.50 per cubic yard; retaining walls, \$1.75 per cubic yard; rip-rap, \$1.25 per cubic yard; clearing and grubbing, \$12 to \$20 per acre; wagon-road bridges, \$21 per 1,000 feet, board measure; trestles and small bridges, \$26 per 1,000 feet, board measure; Howe truss bridges, \$23 per lineal foot; 12-inch terra-cotta pipe, 80 cents per lineal foot; 18-inch terra-cotta pipe, \$1.65 per lineal foot; first-class masonry, \$10 per cubic yard; second-class masonry, \$7.50 per cubic yard; earth overhaul, 1 cent per 100 feet over 800 feet; masonry overhaul, \$1 per mile over one mile; foundations of piers and trestles, at cost, with 10 per cent. added; tunnel ex-

cavation, $4 to $8 per cubic yard, depending on the length ; packing in tunnel, $1 per cubic yard ; timber in tunnel, $35 per 1,000 feet, board measure ; pile driving, 25 cents per lineal foot ; tracklaying, ties, surfacing, etc., $1,350 per mile ; steel rails, about $30 per ton ; iron bridges, 4 to 5 cents per pound, erected.

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